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INVESTIGATION OF UNDERSEEPAGE AND ITS CONTROL,
LOWER MISSISSIPPI RIVER LEVEES. VOLUME I

Army Engineer Waterways Experiment Station
Vicksburg, Mississippi

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CORPS OF ENGINEERS, U. S. ARMY

INVESTIGATION OF UNDERSEEPAGE AND ITS CONTROL

LOWER MISSISSIPPI RIVER LEVEES

COLOR ILLUSTRATIONS REPRODUCED IN BLACK AND WHITE

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THE PRESIDENT, MISSISSIPPI RIVER COMMISSION

CORPS OF ENGINEERS

BY

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

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PREFACE

In September 1940, the Mississippi River Commission initiated a general study of the phenomena of underseepage and its control along Lower Mississippi River levees. This study was prompted by the occurrence of heavy underseepage and sand boils along numerous reaches of these levees during the 1937 flood.

Since initiation of the study in 1940, numerous investigations relating to the problem of underseepage and its control have been made. These studies have included a review and compilation of all underseepage reports made during and since the 1937 high water; exploration and geological studies of numerous sites where underseepage was a serious problem in 1937; installation of piezometers at selected sites to measure substratum pressures beneath and landward of levees; field pumping tests to determine the permeability of the sand aquifer at certain sites; theoretical, electrical-analogy, sand model, and prototype studies of relief wells, partial cutoffs, and landside berms for the control of underseepage; and observation and measurement of natural seepage at certain locations during the 1950 high water.

The studies and data presented in this report are the result of the combined efforts of the Memphis, Vicksburg, and New Orleans Districts and the Waterways Experiment Station, under the general direction of the Mississippi River Commission. Previous underseepage records were compiled by WES; field explorations were made by WES and the above-mentioned Districts; and the piezometer systems were designed by WES in collaboration with the Districts. The Districts installed the piezometers, performed most of the laboratory tests except for certain sites in the Memphis District, made the basic maps and topographic surveys, and read the piezometers during high water. The natural seepage at several sites was measured by the Districts under the direction of WES. The model and theoretical studies, pumping tests, studies of prototype control methods, and preparation of this report were accomplished by WES.

Engineers actively connected with the investigation beginning in 1940 and through 1943 were Brig. Gen. Max C. Tyler, CE, former President,

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MRC; Prof. A. Casagrande, foundation engineer consultant; Prof. S. J. Buchanan, formerly of the MRC; Brig. Gen. K. E. Fields, CE, formerly Director, WES; Mr. William Wells, formerly of WES; Messrs. W. J. Turnbull and C. I. Mansur, WES; Mr. J. B. Eustis, formerly of WES; and Dr. H. N. Fisk, geological consultant. In 1947, a Committee on Underseepage for the MRC was appointed. This committee consists of Messrs. W. J. Turnbull, Chairman, WES; (A. L. Aldridge formerly represented the MRC); J. W. Black, Memphis District; A. C. Williams, Vicksburg District; H. A. Huesmann, New Orleans District; and C. I. Mansur, WES. Other engineers in the Districts actively connected with the installation and reading of piezometers and measurement of seepage are Messrs. J. M. Pollock, C. E. Grimes, and R. V. Bankston. The project engineer in charge of the studies made by WES since 1941 was Mr. C. I. Mansur.

The data were analyzed and this report prepared by Mr. Mansur assisted by Mr. R. I. Kaufman, WES, and Dr. J. R. Schultz, formerly of WES, under the general supervision of Mr. Turnbull. Other engineers and geologists connected with the study at one time or another were Messrs. W. G. Shockley, T. B. Goode, C. R. Kolb, W. B. Steinreide, Jr., and P. R. Mabrey, WES; also, W. F. Jervis, S. J. Johnson, J. A. Focht, Jr., and William Hendy, all formerly of WES.

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NOTATIONS

A Surface area in which seepage emerging landward of a levee is measured

a Well spacing

C Bligh's creep ratio

C_w Lane's weighted creep ratio

c A constant for natural top stratum where $c = \sqrt{\frac{k_b}{k_f\, z_b\, d}}$

c_B Constant for riverside blanket

c_{Bb} Constant for riverside blanket and natural riverside top stratum

D Thickness of pervious substratum

$\bar{D}$ Transformed thickness of pervious substratum

D_n Grain size, n per cent of grains smaller than stated size

D_{10} Effective grain size, 10 per cent of grains smaller than stated size

d Effective thickness of pervious substratum. Depth of cutoff in formulas for partial cutoffs

$d_a, d_b, \ldots, d_n$ Thickness of each stratum comprising pervious substratum

$\bar{d}_n$ Transformed thickness of each stratum comprising pervious substratum

EL_1, EL_2 The extra length of pervious substratum corresponding to the increased resistance to flow into a drainage trench as compared to flow into a fully penetrating, open, vertical, drainage face at the location of the trench

e Void ratio of soil, or base for natural logarithms = 2.718

e_n Natural void ratio of soil

F Factor of safety against uplift at landside toe of levee

G Ratio of flow from a partially penetrating well to that from a fully penetrating artesian well; specific gravity of soil

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H Total net head on levee, or height of flood stage above average low-ground surface, or tailwater, landward of levee

H_{av} Average head (net) in plane of relief wells

H_m Head (net) beneath top stratum midway between relief wells

H_{m_∞} Head (net) beneath top stratum midway between wells in an infinite line of wells

H_{m_N} Head (net) beneath top stratum midway between wells at center of a finite line of wells

H_w Total head loss in a well including elevation head loss

h Effective (net) head acting on a line of relief wells

h_a Allowable (net) head beneath landside top stratum

h_{av} Average head (net) in plane of relief wells measured above H_w

h_c Maximum possible (net) head beneath top stratum; head at which upward gradient through top stratum is equal to critical gradient

h_e Head loss through filter and well screen

h_f Friction loss in well riser and screen = $h_r + h_s$

h_m Head (net) beneath top stratum midway between wells measured above H_w

h_o Head (net) beneath top stratum at landside toe of levee (without seepage control measures) assuming top stratum capable of withstanding such a head

h'_o Head beneath top stratum at landside toe of levee (measured above natural ground surface or tailwater) with a landside seepage berm

h_r Friction loss in well riser pipe

h_s Friction loss in well screen

h_v Velocity head loss in relief well

h_x Head (net) beneath top stratum at distance x landward from landside toe of levee; average head beneath area A in which seepage was measured

$h_{x(c)}$ Net head above ground surface or tailwater at time sand boils or heaving of top stratum occurs

h_1, h_2 Drawdown below water table during a pumping test at distances r_1 and r_2, respectively, from test well. Substratum heads (above ground surface) at two piezometers on a line perpendicular to the levee at distances ℓ_1 and ℓ_2, respectively, from landward toe of levee

Δh Difference in substratum head between two piezometers on a line perpendicular to levee

I Percentage of imperviousness of an area expressed as a decimal

i Upward gradient through top stratum landward of levee

i_c Critical upward gradient through top stratum landward of levee

i_o Allowable upward gradient at landside toe of levee

i_1 Allowable upward gradient at toe of landside seepage berm

K Complete elliptic integral of first kind

k Coefficient of permeability

$\bar{k}$ Permeability of a pervious stratum after transforming substratum to an isotropic, homogeneous stratum

$\bar{K}$ Coefficient of permeability of entire transformed pervious substratum

$\bar{k}_n$ $\bar{k}$ for an individual stratum

k_B Vertical permeability of artificial riverside blanket

k_{Bb} Average combined vertical permeability of riverside natural top stratum and artificial blanket

k_b Vertical permeability of top stratum

k_{bL} Vertical permeability of top stratum landward of levee

k_{bR} Vertical permeability of top stratum riverward of levee, particularly that in riverside borrow pits

k_f Permeability of pervious foundation

k_H Horizontal permeability of a pervious stratum

k_{H-n} Horizontal permeability of individual stratum

k_R Permeability of a remolded soil sample as determined from laboratory tests

k_t Vertical permeability of landside seepage berm

k_v Vertical permeability of a pervious stratum

k_{v-n} Vertical permeability of individual strata

L_B Width of riverside borrow pit and/or required length of artificial riverside blanket

L_s Length of a sublevee basin in ft measured along length of levee

L_1 Distance from riverside toe of levee to river

L_2 Base width of levee, and berm if present

L_3 Landward (effective) extent of top stratum

ℓ Distance between two piezometers installed on a line perpendicular to the levee

ℓ_1, ℓ_2 Respective distances from landside toe of levee (or berm) to piezometers 1 and 2, installed on a line perpendicular to the levee

M Slope of hydraulic grade line at mid-depth of pervious substratum beneath the levee

M_A Slope of hydraulic grade line in pervious substratum at landside edge of (surface) area A

ΔM Net seepage gradient toward a line of relief wells

n Porosity of soil

P Head (net) beneath top stratum midway between wells (same as H_m)

Q Well or seepage flow per unit length of levee

Q_A Rate of seepage flow per unit length of levee emerging in (surface) area A

Q_B Seepage flow into free-draining berm per unit length of levee

Q_r Rainfall runoff into sublevee basin

Q_s Total seepage flow (with or without wells) per unit length of levee per unit of time

Q'_s Seepage flow into sublevee basin

Q_{s-w} or Q_{sw} Natural seepage emerging landward of a line of relief wells

Q_T Total discharge over weir in sublevee

Q_w Flow from a single relief well per unit of time

$Q_{w(100)}$ Well flow per 100 ft of levee

q Well or seepage flow in gpm per 100 ft of levee as determined in model studies

R Maximum average rate of rainfall over an area, in inches per hour, occurring during time of concentration

r Ratio of allowable upward gradient through top stratum at toe of levee to that at toe of seepage berm = i_o/i_1

r_w Effective radius of a relief well

r_1, r_2 Radial distances from a test well

s Distance from landside toe of levee (or berm) to effective source of seepage entry

t Required thickness of landside seepage berm at toe of levee, and height of water in sublevee basin

W Effective length of well screen; penetration of well screen into pervious aquifer expressed as a decimal; base width of levee in formulas for partial cutoffs

$\overline{W}$ Actual length of well screen

X Width of landside seepage berm or sublevee basin

X_I Width of impervious seepage berm

X_{SP} Width of semipervious seepage berm

X_S Width of sand seepage berm

X_P Width of pervious seepage berm collector system

X' Distance in ft from center line of levee to landside sublevee

x_r Effective length of riverside blanket required to reduce h_o to h_a

x_1 Effective length of blanket riverside of levee

x_3 Distance from landside toe of levee (or berm) to effective seepage exit

z Total thickness of top stratum

z_B Thickness of artificial riverside blanket

z_{Bb} Total effective thickness of natural and artificial riverside top stratum

z_b Transformed thickness of top stratum

z_{bL} or z_L Effective thickness of top stratum landward of levee

z_{bR} or z_R Effective thickness of top stratum riverward of levee, particularly that remaining in riverside borrow pit

z_t Critical thickness of landside top stratum

z_1, z_2, z_3 Thickness of individual strata

θ_{av} Average uplift factor for a line of relief wells

θ_m Mid-point uplift factor for a line of relief wells

Σ_p The shortest vertical path of seepage flow around a partial cutoff beneath a levee

γ' Submerged unit weight of soil

γ'_t Submerged unit weight of seepage berm

γ'_z Submerged unit weight of landside top stratum

γ_w Unit weight of water

λ_1, λ_2 Uplift factors in formulas for design of landside drainage trenches

$\$$ Shape factor, the ratio in a flow net of the number of flow channels to number of equipotential drops from the seepage source to exit

SUMMARY

Seepage and sand boils landward of Mississippi River levees have been a problem during major high waters. After the 1937 high water, the Mississippi River Commission initiated a general study of underseepage and various methods of its control. Its specific purposes were to develop a better understanding of the phenomena of seepage beneath levees and of factors influencing underseepage, obtain information that would make possible a rational analysis of underseepage, and to study control methods and develop formulas and criteria for their design.

The studies have included a compilation of past underseepage reports; exploration and geological studies of numerous sites where underseepage was a serious problem in 1937; installation of piezometers at selected sites to measure substratum pressures; field pumping tests to determine the permeability of the sand aquifer; theoretical, model, and prototype studies of relief wells, partial cutoffs, and landside berms for controlling underseepage; and observation and measurement of natural seepage during the 1950 high water.

At several levee sites detailed studies were made to determine the surface geology, characteristics of top stratum and pervious foundation, and conditions riverward of the levee. Lines of piezometers were installed to determine the hydraulic gradient beneath the levee, distance to the effective source of seepage entry and exit, hydrostatic head at landside levee toe at various river stages, and hydrostatic head required to cause sand boils. A relief well system was installed at one site to determine the feasibility and efficacy of relief wells for reducing substratum pressures and controlling underseepage.

From the theoretical, model, and prototype studies it was concluded that:

a. Sand boils and subsurface piping along the Mississippi River levees are the result of excess hydrostatic pressure and seepage through deep pervious strata underlying the levees. The severity of underseepage is dependent upon the head on the levee, source of seepage, perviousness of substratum, and characteristics of the landside top stratum.

b. There is a definite correlation between surface geology and the location and occurrence of underseepage and sand boils.

c. Seepage flow and hydrostatic heads landward of a levee can be estimated from seepage formulas, and/or piezometric data, and a knowledge of riverward and landward foundation characteristics.

d. Piezometer readings obtained during high water provide the best data for determining seepage source and exit, and predicting seepage and hydrostatic heads at a project flood.

e. Removal of the natural top blanket riverward by uncontrolled borrow operations has aggravated the underseepage problem along Mississippi River levees. Except where clay several feet thick was left in place, the source of seepage was in the riverside borrow pits.

f. The permeability of landside top stratum is related to the type of soil, its thickness, and the presence of minute fissures and holes.

g. The permeability of the pervious substratum in the Lower Mississippi Valley ranged from about 500 to 1500 x 10^{-4} cm per sec; it is best determined from pumping tests but can be approximated reasonably accurately from a correlation between D_{10} and k_f.

h. Underseepage can be controlled by properly designed and constructed landside seepage berms, relief wells, and riverside blankets.

As a result of these studies, it is recommended that: the levees in the Memphis, Vicksburg, and New Orleans Districts be investigated with regard to underseepage, using the techniques described in this report; underseepage control measures, where needed, be constructed and maintained; geological and soil conditions having a bearing on underseepage be considered in locating new levees; and cognizance be taken of the effect of borrow operations on underseepage when designing levees.

INVESTIGATION OF UNDERSEEPAGE AND ITS CONTROL

LOWER MISSISSIPPI RIVER LEVEES

PART I: INTRODUCTION

1. Seepage emerging landward of levees in the form of sand boils that actively pipe material poses a threat to the levees' safety during periods of high water. Sand boils and piping during high flood stages are the result of excessive hydrostatic pressure and seepage through deep pervious strata underlying levees in alluvial valleys. Underseepage, when combined with through seepage, may also saturate and reduce the stability of the landside slope of a levee.

2. No crevasses of Lower Mississippi River levees have been positively attributed to sand boils or piping since 1913. However, a failure at Weecama, Louisiana, in 1922 is believed to have been the result of underground piping, and subsurface piping almost caused a levee crevasse at Greenville, Mississippi, in 1929. Excessive seepage and sand boils occurred along numerous reaches of the Lower Mississippi River levees during the flood of 1937. Subsequently many of these levees were enlarged and landside seepage berms and/or sublevees were constructed at known critical seepage areas. However, when these berms and sublevees were designed (1937-40) little information was available regarding the characteristics of the foundation soils, the relation between geological features and underseepage, or rational methods for analyzing subsurface seepage. Because of this lack, the Mississippi River Commission, in September 1940, initiated a general study of underseepage and its control along Lower Mississippi River levees.

Purpose of Study

3. The basic purposes of the investigation were (1) to develop a better understanding of the phenomena of seepage beneath levees and of the factors that influence underseepage, (2) to obtain information that will make possible a rational analysis of underseepage, and (3) to study

means of underseepage control and develop formulas and criteria for their design.

4. More specific purposes of the study were to:

a. Review and compile all underseepage and crevasse data for the Lower Mississippi River levees.

b. Determine the cause of sand boils and correlate geological and soil conditions with the occurrence of underseepage.

c. Determine the geology, the type, thickness, and permeability of the top stratum, and the depth and permeability of the pervious substratum at sites considered typical of conditions existing along Lower Mississippi River levees, and the influence of these features on seepage flow and substratum pressures.

d. Measure the hydrostatic pressure in the pervious substrata beneath and landward of levees during high water at sites with different types and thicknesses of top strata, and from piezometer readings, estimate:

(1) Distance to "effective" source of seepage entry.

(2) Distance to "effective" seepage exit.

(3) Substratum pressures at project flood stage.

(4) Ratio of permeability of the foundation to top stratum.

(5) Upward gradient required to cause sand boils.

e. Determine the effect, from model and field data, of landside seepage berms, cutoffs, and relief wells on the reduction of substratum pressures.

f. Estimate or measure natural seepage emerging landward of levees during high water for comparison with the amount of natural seepage emerging after construction of seepage berms, cutoffs, and relief wells.

g. Develop information and values from the sites studied that will make possible the analysis of seepage conditions and the design of control measures at other similar sites.

h. Consider and develop methods, formulas, and criteria for the design of underseepage control measures.

Scope of General Study and Investigations Made

5. Since initiation of the study in 1940, numerous field, model, and office investigations have been made relating to the problem and control of seepage and substratum pressures landward of levees. As all

of these investigations have been a part of the general study, they are summarized more or less chronologically in the following sections.

Investigation of cause of sand boils at seven sites in the Memphis District

6. The first phase of the general study consisted of an investigation of seven sites in the Memphis District, CE, where excessive underseepage and sand boils had occurred during the 1937 high water. The results of this study, published in October 1941,[35*] showed that the sand boils at these sites can be attributed primarily to the head of water on the levee, the thinness of the landside top stratum, and the existence of a pervious substratum of sand 75 to 150 ft thick which offered relatively free passage beneath and landward of the levee for hydrostatic pressure and seepage from the river and riverside borrow pits. It was concluded that sand boils will occur where the combination of head on the levee, seepage-carrying capacity of the pervious substratum, and characteristics of the landward top stratum are such as to result in the development of a hydraulic gradient through the top stratum greater than the critical gradient required to cause flotation of the soil in cracks, holes, or other weak spots in the top stratum. Because many of the principles and data presented in reference 35 are basic to this report, some of this material is incorporated herein.

Geological investigations

7. The results of the 1941 study indicated the need for more geological information for a proper understanding of factors affecting the occurrence of underseepage. Accordingly, Dr. H. N. Fisk was retained in 1941 by the MRC to make a geological study[9] at five sites[10,11,12,13,14] in the Memphis, Vicksburg, and New Orleans Districts where underseepage had been a major problem during the 1937 high water. Results of these studies, together with a summary report,[15] were submitted during 1942. Some information contained in these reports, together with data obtained

* Raised numerals refer to similarly numbered entries in the List of References at the end of the main report.

in subsequent geological studies, has been utilized in preparing Part II of this report and in developing the geology of 16 typical sites where piezometers were installed for studying seepage beneath levees (see Part IV).

Piezometer sites

8. The study of seven sites in the Memphis District also indicated the need for more specific information and data regarding the development of substratum hydrostatic pressures, the distance from the levee to the effective source of seepage entry, and the relation of these factors to underseepage and sand boils. In order to obtain this information, systems of piezometers were installed in 1942 and 1943 at four of the seven sites (Caruthersville, Commerce, Trotters 51, and Trotters 54). In addition, a system of piezometers was installed in 1943 at Baton Rouge, Louisiana, where some large sand boils had occurred during the 1937 flood.

9. In September 1945, the President, MRC, established a board to determine methods and standards for the utilization of soils data in designing and constructing levees. In April 1947, this board published a "Code for Utilization of Soils Data for Levees," and in a section on underseepage recommended that a number of piezometer systems be installed at strategic locations along the levees for the purpose of obtaining information on substratum hydrostatic pressures considered necessary for planning and designing corrective measures for underseepage. In October 1947, a standing Committee on Underseepage was appointed, consisting of representatives of the Mississippi River Commission, Memphis, Vicksburg, and New Orleans Districts, and the Waterways Experiment Station, to carry out the studies relating to underseepage recommended in the levee code.

10. This committee met in October 1947 to review the adequacy of the previously installed piezometer systems and to select sites for additional systems along the Lower Mississippi River levees. It was the opinion of the committee that the new piezometer systems, together with those previously installed, would indicate the seriousness of the underseepage problem at these sites and make possible a check of theoretical computations and of the results of model studies. When adequate data for sufficiently high river stages were obtained from these sites, the committee thought that the data could, by proper interpretation, be

extrapolated to other sites with similar soils strata and patterns. It was agreed at this meeting that piezometers would be installed at several sites in each district, in addition to those already in existence, so as to cover a range of soil conditions typical of those found along the levees, and that a detailed geological and soils investigation would be made at each of the piezometer sites.

11. These additional piezometer systems were installed in 1948, making a total of 15 systems along the Lower Mississippi River levees and one system on the Red River. Fairly complete piezometer readings were obtained at all of the sites during a high water in 1950.

12. The locations of the piezometer systems are shown in fig. 1 and are as follow:

Memphis District	Vicksburg District	New Orleans District
Caruthersville, Mo.	Upper Francis, Miss.	Kelson, La.
Gammon, Ark.	Lower Francis, Miss.	Baton Rouge, La.
Commerce, Miss.	Bolivar, Miss.	Cotton Bayou, La. (Red River)
Trotters 51, Miss.	Eutaw, Miss.	
Trotters 54, Miss.	L'Argent, La.*	
Stovall, Miss.	Hole-in-the-Wall, La.*	
Farrell, Miss.		

13. The natural seepage emerging landward of the levees was also measured at Gammon, Commerce, Trotters 51, Trotters 54, Stovall, and Baton Rouge sites during the 1950 high water.

Review of underseepage and crevasse data

14. As part of the general underseepage study, a compilation was made of all known crevasse and underseepage data from the records of the Mississippi River Commission and the Memphis, Vicksburg, and New Orleans Districts. This information, together with the locations of all piezometer systems, seepage berms, and permanent sublevees which had been constructed for the control of underseepage, was superimposed on a continuous mosaic made from quadrangle sheets beginning at Cairo, Ill., and extending

* These sites were within the boundaries of the New Orleans District when the piezometer systems were installed.

down the Mississippi River to Baton Rouge, La. Compilation of these data was initiated in 1942 and published in 1948.[39] The data show that underseepage and sand boils were common along many reaches of the levees from Cairo, Ill., to Baton Rouge, La., during the 1937 high water.

15. Detailed field investigations at the locations of levee crevasses known to have been caused by underseepage would have been desirable. However, the conditions that resulted in failure of the levees were destroyed by the scour caused by the crevasse. In view of the elapsed time, lack of precise records, and difficulties involved in quantitative analyses, no investigations were made at old crevasses.

Sand model studies of relief wells

16. A number of sand models were also constructed to study the phenomenon of underseepage and its control by means of relief wells. More specific purposes of these model studies were to investigate the operation of relief wells and to observe well and seepage flows, and landward substratum hydrostatic pressures with and without relief wells in operation for various foundations, seepage entrances, and landward top strata. The conditions studied were considered to represent at least qualitatively conditions commonly encountered in the Lower Mississippi River Valley.

17. In the models, relief wells with proper spacing and penetration effectively reduced excess hydrostatic pressure landward of levees underlain by a pervious foundation for a wide range of seepage entrances, foundation conditions, and landward top strata. With adequate well spacing and penetration, uncontrolled seepage normally emerging landward of a levee (without wells) was materially reduced, although the total underseepage flow was increased by about 20 to 40 per cent for a model typical of conditions along Lower Mississippi River levees. The results of these studies were published[43] in 1949 and are summarized in Appendix A.

Sand and electrical-analogy
model studies of partial cutoffs

18. In 1946 the Office, Chief of Engineers, and the Mississippi River Commission requested the Waterways Experiment Station to study the effect of partial cutoffs, installed along the riverside toe of a levee,

on underseepage and hydrostatic pressures beneath and landward of a levee. Various foundation and seepage entrance conditions were selected for study as representing, at least qualitatively, certain limiting conditions commonly encountered in the Lower Mississippi River Valley. The methods of analysis used included sand and electrical models, graphical analyses, and mathematical computations. These studies showed that partial cutoffs had relatively little effect on the reduction of underseepage or substratum hydrostatic pressures for the conditions tested. The results of the investigation of partial cutoffs were published[40] in 1949 and are summarized in Appendix B.

Field installations of relief wells

19. In December 1942 and January 1943 lines of experimental relief wells were installed at Commerce, Trotters 51, and Trotters 54, Miss., and at Wilson Point, La., where underseepage had been a problem during the 1937 flood and where foundation explorations had already been made. At Commerce the wells were installed to permit determination of the effectiveness of different degrees of screen penetration into the underlying pervious aquifer. Piezometers had previously been installed at the four sites to evaluate the performance of the well systems. The systems operated during the high water in 1943, but the wells were found to be too small in diameter and subsequently were plugged or pulled. However, valuable information pertaining to the permeability of the foundation, well flow, and pressure reduction was obtained, and was published for the Commerce and Trotters systems in June 1950.[45]

20. In the summer of 1950 a new well system and additional piezometers were installed at Trotters 54, Miss., for the purpose of making a full-scale field test of the efficacy of a larger capacity relief well system for controlling underseepage. This well system operated very satisfactorily during the high water in 1951 and 1952. Piezometer readings and seepage observations made during both of these high waters indicated that the well system reduced substratum hydrostatic pressures landward of the levee to a small fraction of the head on the levee, and also intercepted a large portion of natural seepage which otherwise would have emerged landward of the levee. The design of this well

installation and an analysis of its operation during the 1951 and 1952 high waters were reported[51] in April 1952 and February 1954 and are summarized in Appendix D.

Field installation of partial cutoff

21. Another phase of the investigation of the control of underseepage consisted of the development of a machine and procedures for installing an impervious partial cutoff 30 feet deep or more along the riverside toe of a levee. This project was carried out by the Memphis District, CE, during 1946-1951 and culminated in the completion of a 40-ft cutoff along a 1400-ft reach of levee at Trotters 51, Miss., after which the project was discontinued. No performance data are available for the cutoff installed.

Seepage berms

22. The most commonly used method for safeguarding levees along the Lower Mississippi River against the hazards of sand boils and subsurface erosion is the construction of seepage berms along the landside toe of the levees. At the time when most of these berms were designed and constructed (1937-1940) little information was available regarding the characteristics of the foundation soils; neither were formulas or methods available for designing such berms. As a result, most of these seepage berms were designed on an empirical basis. Subsequently, more rational procedures and formulas have been developed for the design of seepage berms based on seepage formulas[3] and electrical analogy model studies performed by the Kansas City District, CE. No data are available regarding the substratum pressures that existed prior to construction of seepage berms to compare with pressures measured by piezometers installed after the berms were constructed.

Field pumping tests

23. Data obtained from the relief wells at Commerce, Trotters 51, and Trotters 54, Miss., during the high water in June 1943 revealed that the flow from the wells was greater than anticipated in the original design of the systems.[45] This greater flow was attributed to a more pervious foundation and/or a closer seepage entrance on the riverside of the levee than had been assumed in the design. Because of the importance

of accurate knowledge of the quantity of flow in the design of a well system, field pumping tests were conducted at Commerce for the purpose of determining more precisely the over-all permeability of the pervious substratum. These tests were initiated in the fall of 1943 and were completed in the spring of 1944.

24. During installation of the new well system at Trotters 54 in 1950, pumping tests were performed to determine the flow for various drawdowns in the well, head loss through the filter and well screen, and the permeability of the pervious sand stratum. The results of these pumping tests were reported[51] in April 1952. Since then comprehensive pumping tests have been made on wells installed along the levees in the St. Louis District, CE,[54] and at the site for a lock and a control structure to be built in conjunction with the control of Old River south of Natchez, Miss. The results of these field pumping tests are summarized in Appendix C.

Scope of This Report

25. Part II of this report presents a discussion of the geology of the Lower Mississippi River Valley as related to underseepage, and defines and describes the various types of geological features encountered in the valley that have an effect on underseepage. Part III is a discussion of the phenomena of underseepage, sand boils, and piping, and of foundation characteristics and factors related to the development of underseepage and substratum pressures. This part also presents the basic methods and formulas for analyzing underseepage as commonly experienced along the levees in the Mississippi Valley.

26. Part IV presents the studies and analyses made at the piezometer sites, and includes a description of soil conditions, history of underseepage, description of control measures that have been constructed, and detailed analyses of all piezometric and seepage data obtained to date. Volume 2 of this report presents the following information pertaining to each site: maps, seepage data, geological and soil profiles, laboratory data, levee grades, piezometer readings and river stages recorded during high water, plots of the hydraulic gradient beneath the levees and

landward of the levees at selected river stages, and plots of individual piezometer readings vs river stage for selected piezometers at the land-side toe of the levees.

27. An evaluation of data from all sites and a comparison of geologic conditions with seepage observations are presented in Part V. Methods of underseepage control together with formulas and graphs for the design of seepage berms, relief wells, toe drains, and riverside blankets as developed to date are presented in Part VI. Design, construction, and installation of seepage control measures are discussed in Part VII; maintenance and observation of seepage control measures and piezometers are discussed in Part VIII. Where additional underseepage control measures are considered necessary at the sites studied, the design of such measures is illustrated in Part IX. Results of the investigation are given in Part X and conclusions and recommendations in Parts XI and XII. A list of references is included at the end of the main report.

28. Model studies of relief wells and cutoffs, field pumping tests, and observations of a relief well system at Trotters 54 are summarized in Appendices A-D.

PART II: GEOLOGY OF THE LOWER MISSISSIPPI RIVER VALLEY AND ITS INFLUENCE ON UNDERSEEPAGE

29. Geological studies of several sites along Lower Mississippi River levees in 1941 and 1942 showed a definite correlation between the distribution of alluvial deposits of sand, silt, and clay with the location and occurrence of underseepage and sand boils.

30. The geology of the Lower Mississippi River alluvial valley is now fairly well known and has been discussed in several rather comprehensive reports.[16,17,30] It is therefore unnecessary to present a detailed account of the subject, and in the following pages emphasis is placed on those aspects that are of greatest importance to underseepage. The purpose of this part is to review the general geology of the Lower Mississippi River Valley and to illustrate the relation of geological deposits to underseepage. Another purpose is to define terms and describe geological features so as to make possible a clearer understanding of the descriptions of the geology of the specific underseepage areas investigated which are given in the discussions of individual sites (Part IV).

The Alluvial Valley of the Lower Mississippi River

31. The alluvial valley of the Lower Mississippi River begins near the confluence of the Mississippi and Ohio Rivers, Cairo, Ill., and extends to the Gulf of Mexico (see fig. 1). The valley varies in width from about 30 miles at Natchez, Miss., to 125 miles between Memphis, Tenn., and Little Rock, Ark.; the ground surface in the valley has an average slope toward the Gulf of about 0.6 ft per mile.

32. The alluvial deposits in the Lower Mississippi River Valley fill a trench or canyon having all the characteristics of a former stream valley and ranging in depth from about 100 ft in the upper part of the alluvial valley to 400 ft near the Gulf. The origin of this buried canyon is attributed to changes in sea level caused by the glaciers which were formed during the Pleistocene epoch. It is estimated that

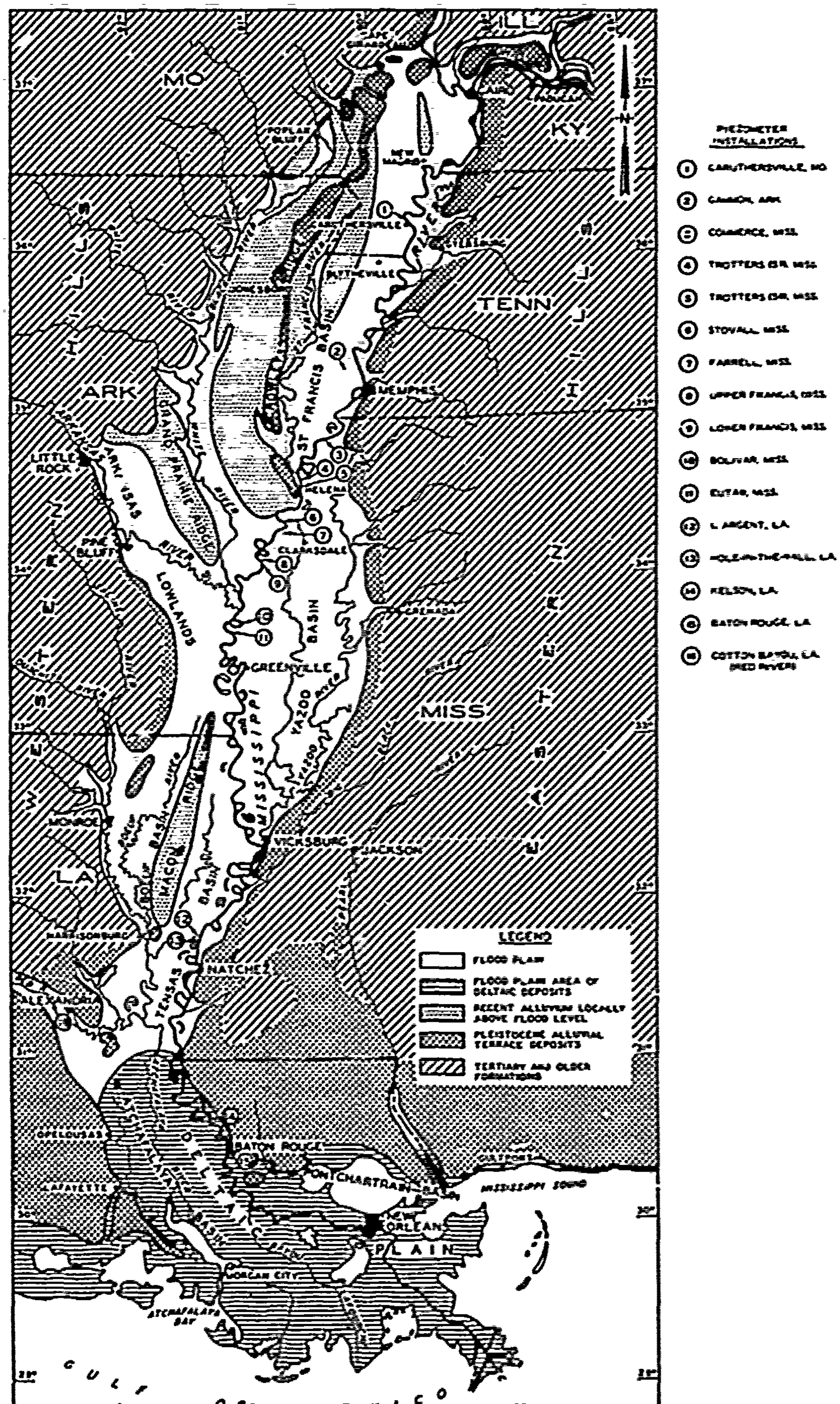


Fig. 1. Plan of alluvial valley of the Mississippi River and location of piezometer installations

when the glaciers reached their maximum extent sufficient water was withdrawn from the ocean to lower sea level by about 400 ft. In response to a greatly increased slope, the Mississippi River and tributaries were able to cut their valleys to depths adjusted to the lowered sea level; when the sea level rose as the glaciers retreated these valleys were filled by alluvial deposits. This cycle of alternating advance and retreat of the ice took place at least four times during the Pleistocene epoch. The entrenched valley proper was formed during the late Wisconsin glacial stage, some 60,000 to 30,000 years ago, when the ice advanced southward for the last time. A somewhat idealized illustration of the entrenched valley is presented in fig. 2.

33. The river has meandered widely over the alluvial valley, leaving behind a series of typical features, such as oxbow lakes and natural levees, which indicate its former presence. The time required to change from one course to another has been carefully studied, and Fisk[16] has divided the former courses of the river into 20 stages, separated by intervals averaging 100 years in duration, the earliest course being represented by the number 1 and the present course by the number 20. Still earlier stages have been represented by letters. This system of designating former river courses has been adopted on the geologic maps of all Mississippi River underseepage sites investigated (see vol. 2).

The Alluvial Fill

34. As the glaciers of the late Wisconsin stage began to melt, some 30,000 years ago, the sea level gradually rose to its present position causing the entrenched valley to become filled with a series of sandy gravels, sands, silts, and clays that can be grouped into two broad units: (a) a sand and gravel substratum, and (b) a fine-grained top stratum. These units are of basic importance to the underseepage problem and, therefore, require fairly detailed description. The alluvial materials are generally underlain by relatively impervious Tertiary deposits.

The pervious substratum

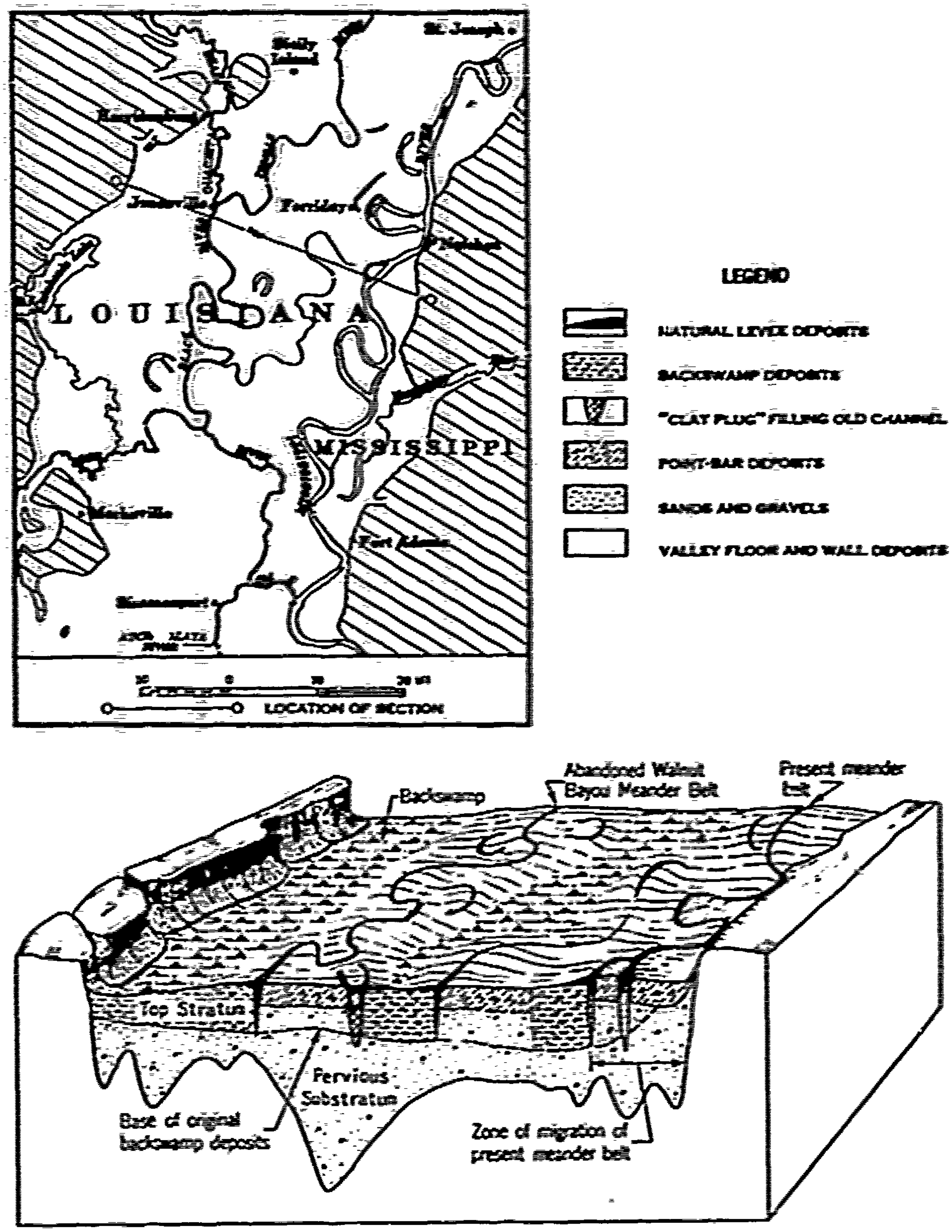
35. Gradual rise of sea level accompanying final retreat of the

ice affected the lower courses of the Mississippi River before its upper reaches, and the main channel before its tributaries. It became, in consequence, an overloaded braided stream in which large quantities of gravel-bearing sands were deposited. As sea level continued to rise and the deposits on the floor of the entrenched valley continued to thicken, stream slopes were progressively reduced and both the quantity and grain size of the materials transported by the river decreased. The gravel-bearing sands were succeeded by coarse sands grading upward into progressively finer materials and terminating in deposits of very fine sand. This upward gradation from coarse to fine materials reflects a gradual adjustment between the transporting capacity of the river and available load. Fine to very fine sands were not deposited until sea level had reached essentially its present stand and the river began to change from a braided to a meandering stream.

36. The sandy alluvium is 180 ft thick near Sikeston, Mo.; 100 ft thick at Memphis, Tenn.; about 150 ft thick in the latitude of Yazoo City, Miss.; and in the latitude of New Orleans, La., thicknesses in excess of 200 ft are not uncommon. North of the Louisiana-Arkansas boundary, medium sands are within 5 to 20 ft of the ground surface, but south of this line the thickness of the overlying materials increases and in the vicinity of Houma, La., "clean" sands are over 50 to 100 ft below the surface. The thickness of the clean sand substratum ranges from 75 to 150 ft. The uneven thickness is mainly a result of irregularities in the lower surface of the valley floor. A typical section of the substratum is shown in fig. 2.

The relatively
impervious top stratum

37. Some 6000 years ago, sea level reached essentially its present position, and the sedimentary load carried by the river soon became substantially adjusted to the slope and velocity. Rapid filling of the entrenched valley ceased, and the river began to show no marked tendency either to cut away or fill in its valley, a condition often spoken of as "poised" or "graded." The former braided channel was replaced by meanders swinging in wide curves, but seldom escaping from a meander



Present Stage-Mississippi, diverted from Walnut Bayou Meander Belt west of Vicksburg, established present course in lowland east of the Cocodrie Meander Belt Ridge.

Fig. 2. Block diagram of alluvial valley of the Lower Mississippi River

belt about 10 miles wide. Characteristic sediments consisting of point bar, channel fill, natural levee, and backswamp deposits, were laid down in meander belts. These deposits are marked by both lateral and vertical discontinuities and wide disparity in grain size and permeability. They are, therefore, of primary importance to the study of underseepage, and the chief contribution of geology to this study lies in the accurate delineation of the various types of meander belt deposits.

38. Point bar deposits. As meander loops increase in radius by erosion of the concave bank, deposition takes place on the convex bank where low sandy ridges are built up. The elongated depressions between ridges are known as swales and usually become filled with fine-grained deposits. The alignment of ridges and swales tends to parallel the channel in which they were laid down, but owing to downstream migration of meanders successive ridges and swales tend to overlap and truncate each other in a complicated fashion. The convex banks of meander loops are known as "points" and the aggregate of ridges and swales found on a point is often referred to as a "point bar accretion." The sands composing the ridges between swales are generally cross-bedded and often grade downward into the underlying clean sands. The upper part of these ridge and swale deposits usually consists of relatively impermeable silty sands, clay silts, and silty clays that are laid down during gradual migration of the river channel and the zone of active sedimentation away from the ridges. This covering of point bar deposits by a blanket of silts and clays is commonly found in the Lower Mississippi River Valley. Nevertheless, the sandy ridges are the most permeable of the top stratum materials, and usually exhibit heavier underseepage than adjacent areas. The swales generally become filled with deposits of silts and clays. Swale fillings range in width from 20 to 500 ft, but in some instances may attain widths of 1500 ft or more. Their thickness usually ranges from 5 to 40 ft, but thicknesses of 80 to 100 ft are known. In comparison with the sandy ridges, swale fillings are practically impervious. Typical examples of point bar deposits exist at Commerce and Lower Francis piezometer sites (see plates 33, 39, 142, and 144).

39. Included within the category of point bar deposition are the

so-called channel bars. These are elongate accretion deposits built up as a river reach migrates laterally. In contradistinction to the typically arcuate accretion deposits built by a meandering river bend, channel bar deposits are essentially straight or only slightly sinuous. An excellent example is seen at the Trotters 51 site (plates 62 and 67). Channel bars, from an underseepage point of view, are analogous to the sandy ridges found on point bars. The swale fillings of channel bars tend to be somewhat longer and deeper than those found on point bars but are otherwise similar.

40. Channel-fill deposits. As a result of downstream migration of meanders, aided by local inequalities in erosional resistance of the banks, opposite arms of a meander loop may meet and form what is known as a cutoff. A meander loop separated from the main channel by a cutoff soon becomes plugged at both the upper and lower arms, and the central portion is converted into one of the familiar oxbow lakes. Sediments filling the central portions of the loop are generally clay in contrast to the somewhat sandy and silty materials plugging the upstream and downstream arms. Examples of clay-filled former river courses can be found at Upper Francis and L'Argent (see plates 127 and 182). The width of a clay plug is roughly equal to that of the former channel and the thickness may approach or even equal the depth of the former river channel. A channel fill in the Memphis District is 90 ft thick; the False River fillings north of Baton Rouge, La., are about 125 ft thick; and an abandoned channel deposit near Plaquemine is 140 ft thick.[17] True clay plugs are practically impermeable and no important underseepage has been known to penetrate them.

41. An exceptionally wide and deep swale may become enlarged under certain conditions until it becomes the main channel of the river, forming a "chute cutoff" as contrasted with the neck type of cutoff described above. In most chute cutoffs, flow continues through the old channel for a considerable time but its upper end eventually becomes plugged by sandy materials. The lower end of the old channel may receive considerable amounts of fine-grained materials which form a relatively impermeable plug. The fillings of channels abandoned by chute cutoffs may

consist of either silts or clays, whereas those formed by neck cutoffs are usually filled with clay. Furthermore, the fine-grained materials filling chute cutoffs are usually somewhat thinner than those filling neck cutoffs. Consequently, chute cutoffs seldom lead to the formation of as effective underseepage barriers as those formed in connection with neck cutoffs.

42. It is pointed out that types of deposits and depth to clean sand within abandoned channels may vary considerably, depending on hydraulic conditions subsequent to cutoff. Cutoff channels, which for a number of reasons tend to stay open for a considerable period after cutoff, generally fill with coarser grained material. Where complete cutoff is effected in a short period of time (5 years or less) the abandoned channel filling tends to be deep and fine grained. The latter situation is overwhelmingly predominant in the case of neck cutoffs; less predominant in the case of chute cutoffs.

43. Natural levees. When the river overtops its banks the water spreads out, the velocity is checked, and deposition of a portion of the load results. In this manner, long ridges known as natural levees are formed on the outside of meander loops and along both banks of straight reaches. The width of natural levees bordering the present and past courses of the Mississippi River is extremely variable and ranges from less than one-fourth mile to over four miles. Their thickness adjacent to the riverbanks along which they were laid down ranges from 5 to 15 ft in the northern part of the valley, and is as much as 20 ft in the latitude of Baton Rouge, La. Their height above the surrounding flood plain ranges from 5 to 10 ft. At the time of deposition the crest is adjacent to the riverbank with a short steep slope toward the river, and a long, gradually decreasing slope toward the adjoining low-lying areas away from the river. The landside slopes may often be traversed by a network of intercommunicating channels formed by crevasses during exceptionally high floods. The aerial photograph pattern of natural levees and associated crevasses is well illustrated in the mosaic of the Trotters 51 site (see plate 64).

44. The coarsest materials, ranging from sandy silts to silty

clays, are dropped nearest the river, and the grain size decreases progressively with distance from the banks. In other words, natural levee deposits grade landward into comparatively uniform clays that are indistinguishable from those laid down in the adjoining lowlands, or backswamps, and for this reason the boundary between natural levee and backswamp deposits can seldom be fixed with any degree of accuracy. The stratification of natural levee deposits is essentially horizontal. Those found in the northern part of the valley are mainly sandy silts and silty clays; in the southern portion they consist chiefly of clays and silty clays. According to Fisk,[17] the grain size is generally coarser than the average backswamp and channel fill material, but somewhat finer than the fine-grained upper portion of point bar ridges. Crevasse channels in natural levees may be filled with silts and silty clays that are somewhat more permeable than the main body of the deposit.

45. Natural levees are usually comparatively well drained, both internally and externally, and consequently have generally been cleared for farming. They are thus fairly easy to distinguish on aerial photographs and topographic maps, but are extremely difficult to delineate in a manner showing their relationship to underseepage. It was decided, after considerable discussion and experimentation, to show on the geologic maps only the better-defined natural levees and, except where they are notably sandy or silty, to omit a separate designation on the geologic profiles. As a result, the boundaries of natural levee deposits shown on the maps and soil profiles generally do not coincide. Examples of natural levee deposits are shown in many of the plans and soil profiles of the piezometer sites.

46. Backswamp deposits. The low-lying areas on the landside of natural levees are known as backswamps (see Fig. 2). These areas receive only quiet floodwaters; therefore, the sediments laid down consist mainly of thinly laminated clays and silty clays with minor amounts of silts. The stratification is essentially horizontal. The thickness ranges from 15 to 20 ft in the vicinity of Helena, Ark., to 20 to 70 ft in southern Louisiana where the backswamp deposits merge with interfingering masses of swamp, marsh, and near-shore marine deposits of

clays, silts, and sands forming the deltaic plains. Backswamps have poor surface drainage and the soils are normally saturated the year round. As a result, they are seldom farmed. Backswamps are readily distinguished on aerial photographs by their tree cover and lack of systematic drainage pattern. Significant underseepage through thick backswamp deposits has never been observed.

Influence of Geology on Underseepage

47. The emergence of seepage landward of a levee is influenced by: (a) configuration of geological features such as swale fillings and clay plugs and their relation to the levee; (b) characteristics and thickness of the top stratum; (c) cracks and fissures formed by drying or other natural causes; (d) borrow pits, post holes, seismic shot holes, and other works of man; and (e) decay of roots, uprooting of trees, animal burrows, crayfish, and other organic agencies. The severity of underseepage along a reach of levee is frequently dependent upon the configuration of geological features in the area, as discussed in the following subparagraphs.

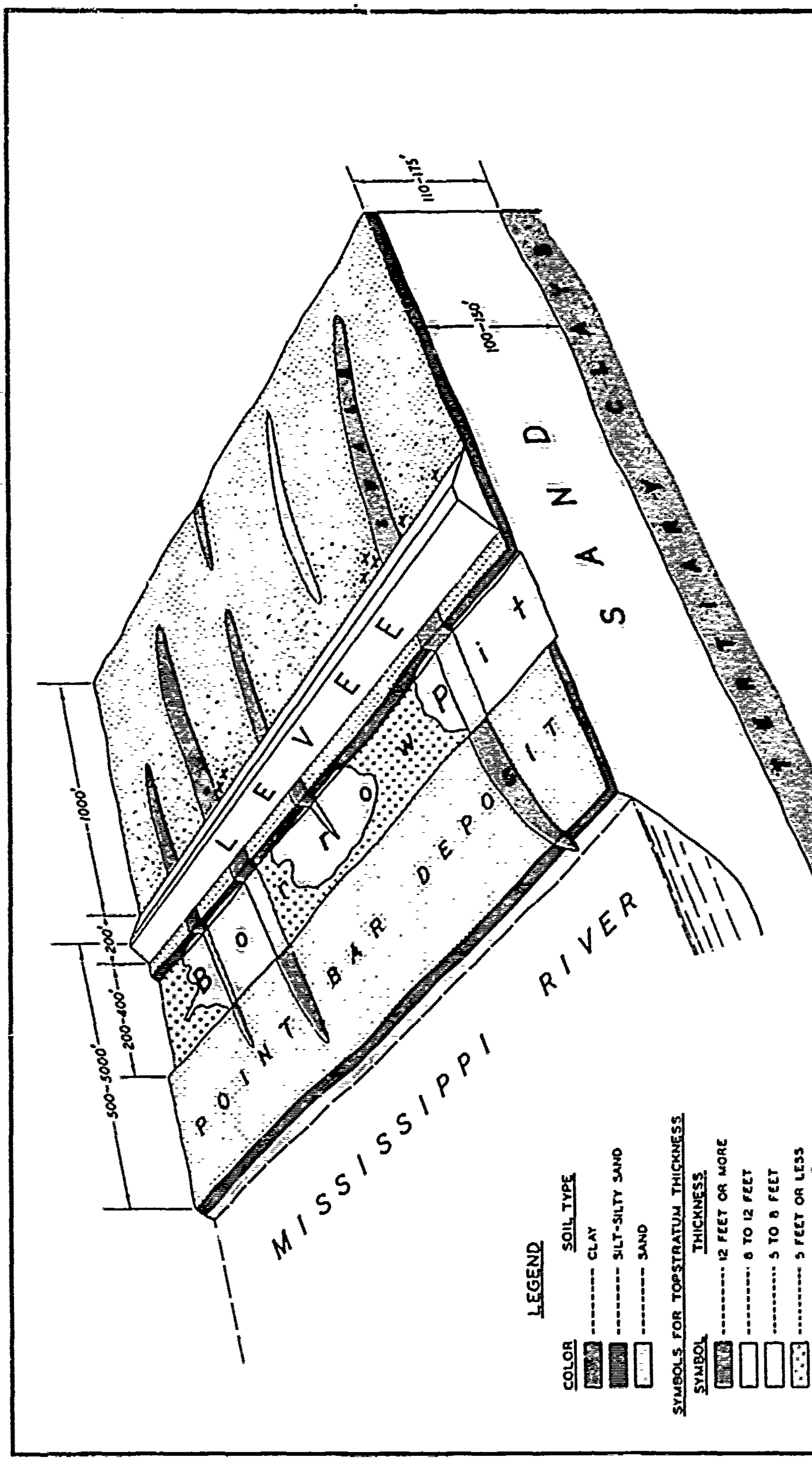
a. Point bar deposits. Point bar deposits are the most heterogeneous of all the materials forming the top stratum. Fine-grained swale fillings of highly variable thickness separating sandy ridge deposits are the chief discontinuities found in point bar materials. Inasmuch as the ridges are considerably more pervious, practically all of the underseepage flows up in the ridges regardless of the orientation of the swales to the levee. The greatest concentration of seepage always occurs along the edges of swales and at the landside levee toe, as illustrated in fig. 3. In instances where the long dimension of the swales is parallel to or intersects the levee toe at a small angle, seepage is particularly concentrated in the sandy ridges where the edges of the swales intersect the levee toe (see fig. 4). Sharp inside angles or bends in levee alignment cause a greater concentration of flow, as a three-dimensional flow pattern results with all flow lines directed toward the angle at the landside toe.

b. Clay plugs and channel fillings. Clay plugs and channel fillings differ from swale fillings mainly in their greater width and thickness. They are also generally longer, and are often somewhat more uniform in composition.

Their effect on the distribution of underseepage is similar to that of swale fillings but, owing to their greater thickness and width, is considerably accentuated (see figs. 5 and 6). The worst possible underseepage condition is found where a deep or wide clay-filled channel exists parallel to and immediately landward of the levee toe. In a case of this type, all of the underseepage is concentrated in the small area between the levee toe and the clay plug. However, clay-filled channels that parallel the levee reduce underseepage if the landside toe of the levee overlaps the filled channel. Such channels immediately riverward and overlapped by the levee also tend to reduce underseepage by forming a relatively impervious riverside blanket.

c. Natural levees. Where the levee is founded on top of a continuous natural levee deposit of silts or silty sands underlain by clay, the natural levee deposit will act as a semipervious aquifer for shallow underseepage. However, such deposits do add to the weight of the underlying clay and thereby help resist lifting of the top stratum by excessive substratum pressures in underlying clean sands. Sandy or silty crevasse fillings may also act as seepage channels, but because of their narrow cross section and irregular shape any underseepage originating in these fillings is very restricted in distribution.

d. Backswamp deposits. As previously mentioned, these materials are generally thick and practically impervious, and no significant underseepage is known to penetrate them. However, if for any reason the continuity of backswamp clays is broken, or their thickness sufficiently reduced by excavations such as drainage ditches or borrow pits, underseepage and possibly sand boils may be expected to develop.



SEEPAGE THROUGH POINT-BAR DEPOSITS OF SILTS AND SILTY SANDS WITH OCCASIONAL CLAY-FILLED SWALES

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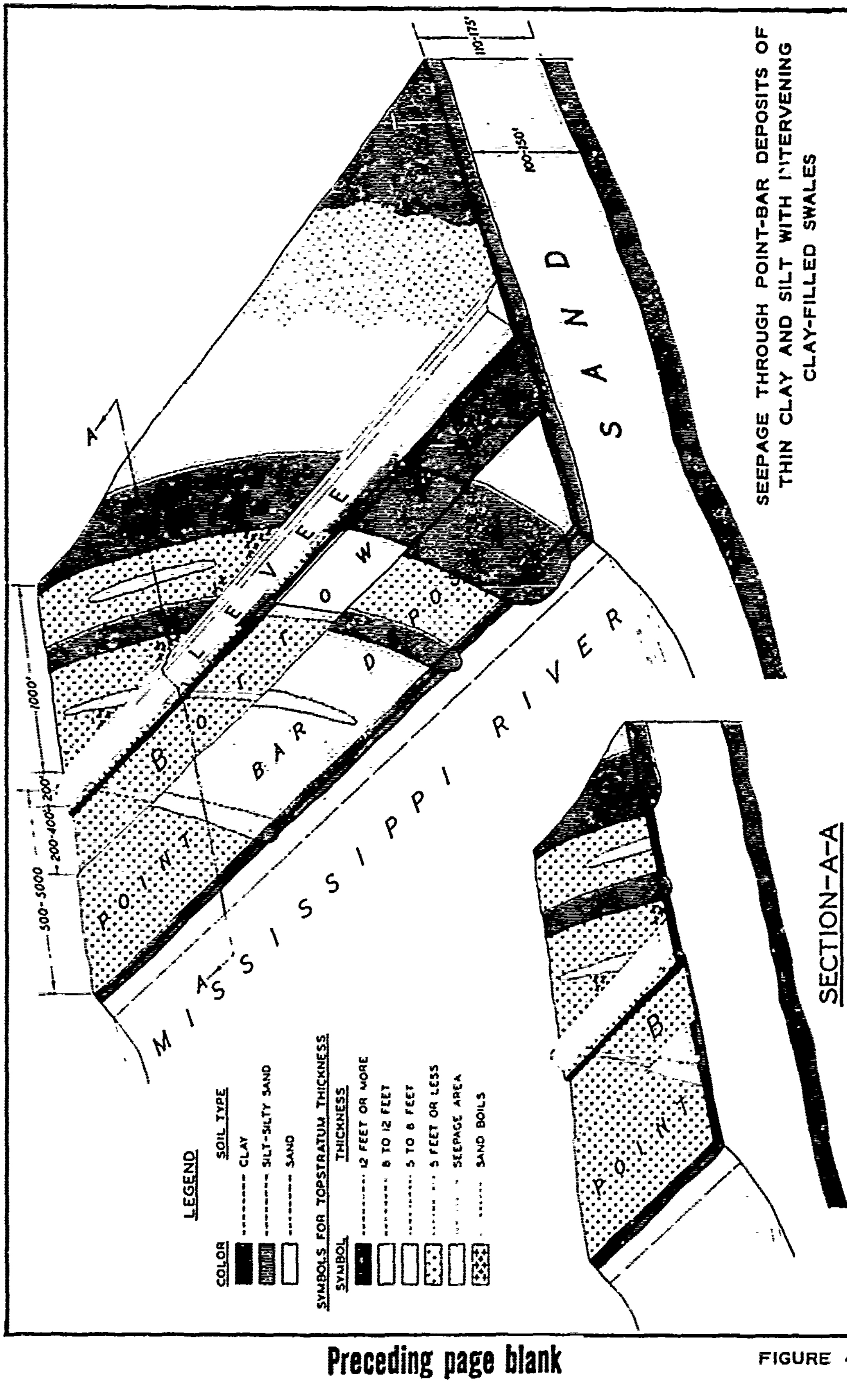
FIGURE 3



SEEPAGE THROUGH POINT-BAR DEPOSITS OF THIN CLAY AND SILT WITH INTERVENING CLAY-FILLED SWALES

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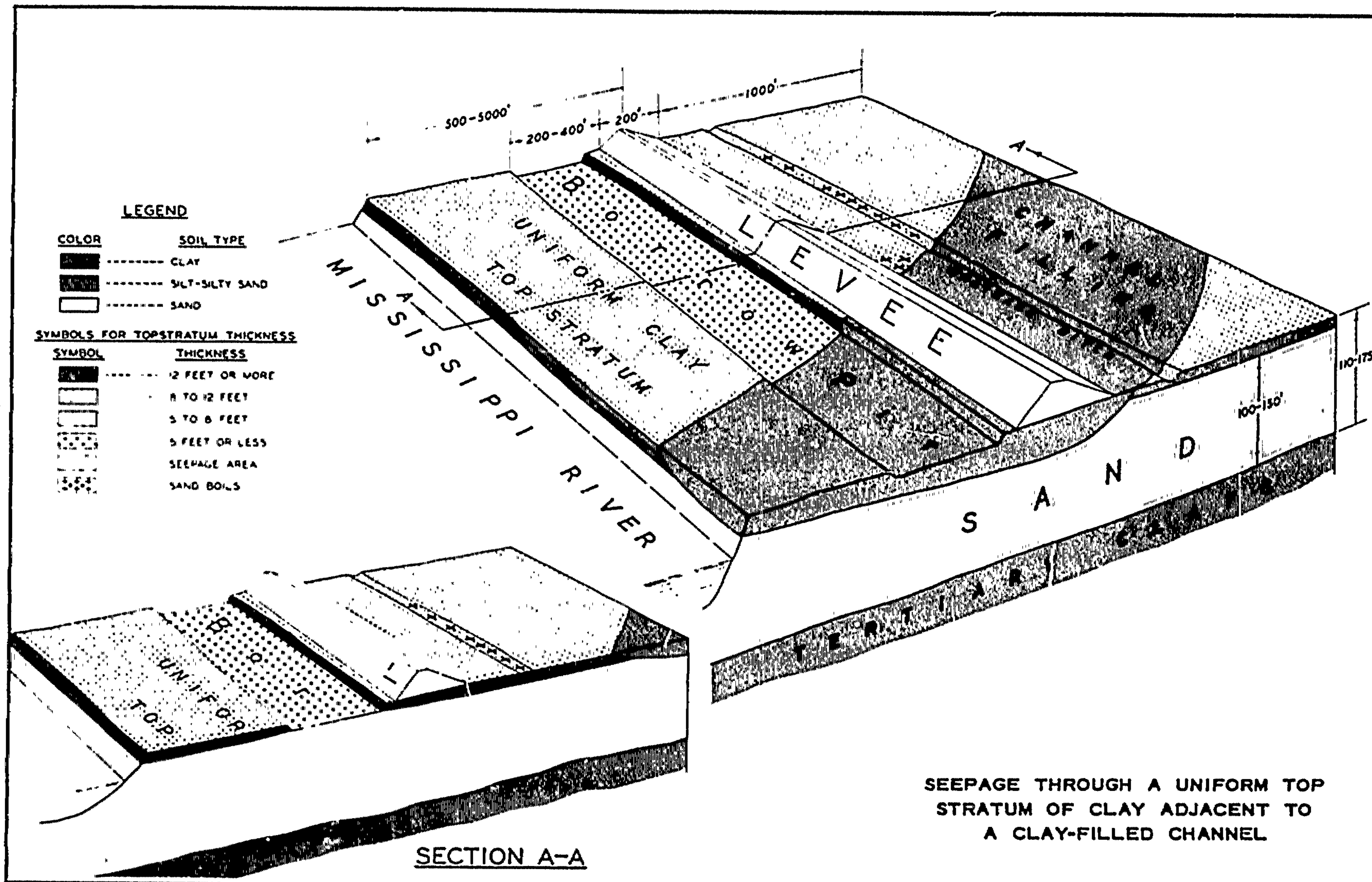
FIGURE 4



SEEPAGE THROUGH A UNIFORM TOP STRATUM OF CLAY ADJACENT TO A CLAY-FILLED CHANNEL

FIGURE 5

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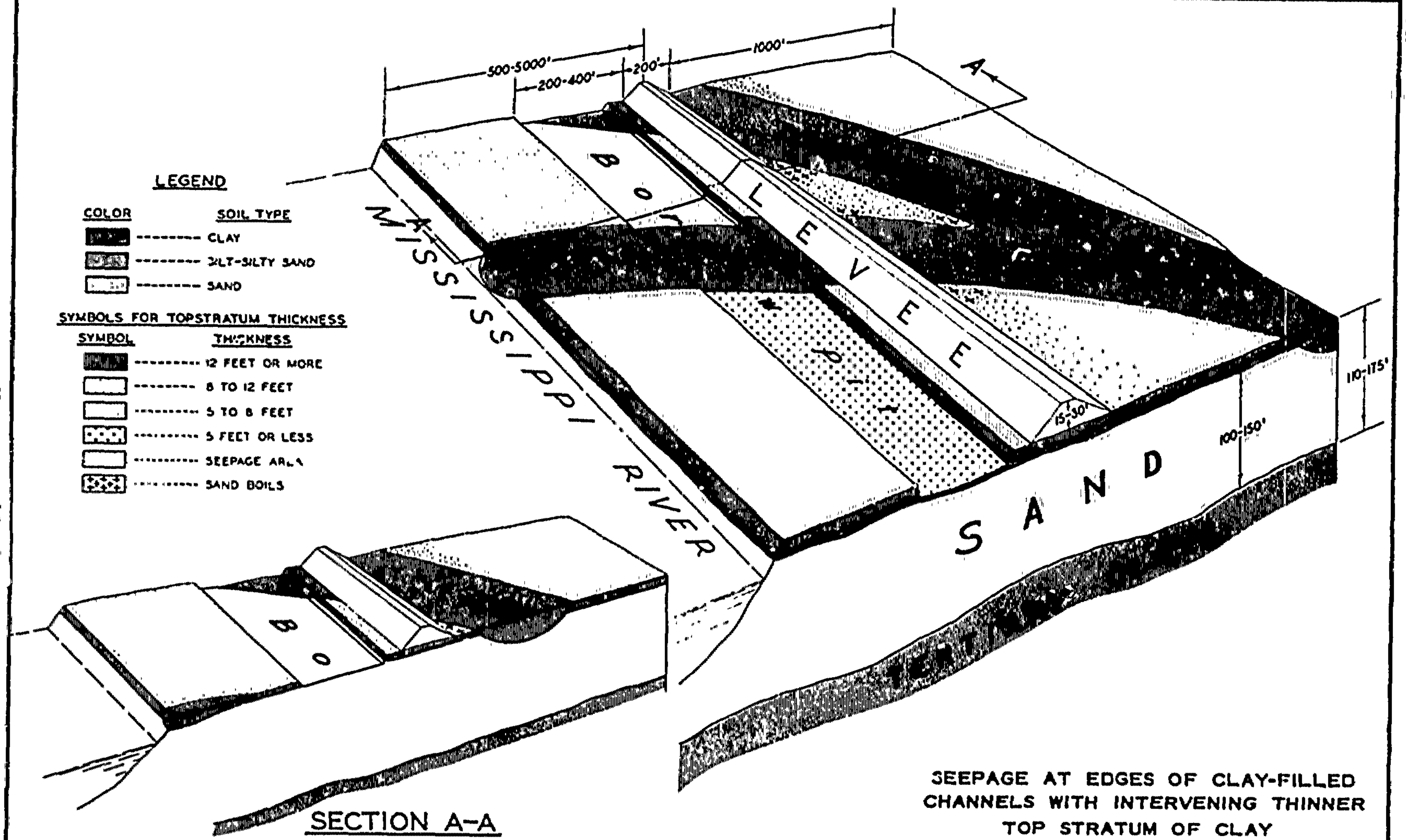

SEEPAGE AT EDGES OF CLAY-FILLED CHANNELS WITH INTERVENING THINNER TOP STRATUM OF CLAY

FIGURE 6

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PART III: OCCURRENCE AND ANALYSIS OF UNDERSEEPAGE

Development of Underseepage and Sand Boils

48. When a levee is subjected to a differential hydrostatic head of water as a result of river stages being higher than the adjacent land, seepage entering the pervious substratum through the bed of the river, riverside borrow pits, and/or the riverside top stratum, creates an artesian head and hydraulic gradient in the sand stratum under the levee. This gradient causes a flow of seepage beneath and landward of the levee as illustrated in fig. 7. Such seepage emerging at or landward of the levee toe is generally termed underseepage. If the hydrostatic pressure in the pervious substratum landward of a levee becomes greater than the submerged weight of the top stratum, the uplift pressure will cause heaving of the top blanket and it may rupture at weak spots with a resulting concentration of seepage flow in the form of sand boils. Active erosion of subsurface material as a result of substratum pressure and concentration of seepage in localized channels is termed piping. Where the foundation and top strata are heterogeneous, as is usually the case, seepage tends to localize instead of causing the entire top stratum to heave or become "quick." A combination of excess head and seepage can create sand boils and subsurface erosion that may culminate in formation of piping beneath the structure with no heaving action. Terzaghi[29] has stated that the mechanics of this type of piping defy a theoretical approach. Fig. 8 shows examples of small sand boils, and fig. 9 illustrates some rather serious piping.

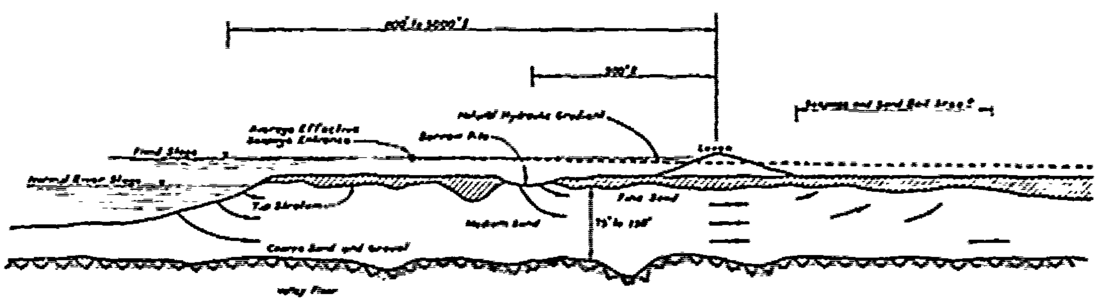

Fig. 7. Generalized cross section of geologic strata and levee

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Fig. 8. Small sand boils in sack sublevee basin near Friars Point, Miss., 1937 high water

Fig. 9. Sand boil at edge of slough 200 ft from levee toe at Stovall, Miss., discharged approximately 1000 cu yd of sand during 1937 flood

49. The hydraulic gradient required to cause heaving is called the "critical hydraulic gradient." It is the ratio of the submerged unit weight of the soil to the unit weight of water, and is usually about 0.7 to 0.85. Any tendency for the hydraulic gradient to increase above the "critical" gradient only causes additional sand boils or increased percolation. High exit gradients and concentrations of seepage are usually found along the landside levee toe, at thin or weak spots in the top stratum, and adjacent to clay-filled swales or channels. In these cases the exit gradient may become critical at singular points, while the average gradient through the top stratum is still less than the critical value.

50. Where seepage is concentrated to the extent that turbulent flow is created, the flow will cause erosion in the top stratum and development of a channel down into the underlying silts and very fine sands which frequently exist immediately beneath the base of the top stratum. As the channel increases in size and/or length, a progressively greater concentration of seepage flows into it with a consequent greater tendency for erosion to progress beneath the levee. This is especially true if the top stratum is cohesive and the underlying soils are susceptible to erosion. Shrinkage cracks, root holes, and holes made by man or burrowing animals form ready channels in the top stratum for development of localized flows and attendant piping.

51. If there were a completely impervious landside stratum overlying the pervious foundation and no flow of water landward, the hydrostatic pressure head beneath the top stratum landward of the levee would equal the river stage. This condition seldom exists, as generally some water is flowing landward through the pervious foundation and upward through the surface stratum with a consequent landward decrease of pressure head (see fig. 7). This dissipation of head has been observed in model studies and at the piezometer sites, and partially explains why sand boils have not occurred in some areas where, without such head loss, subsurface pressures during high-water stages would have been sufficiently great to cause heaving or sand boils landward of the levee. The reduction of the hydrostatic pressure beneath the top stratum as a result of

natural seepage should not be considered as a guarantee against sand boils or subsurface erosion, since even with such reduction the residual substratum pressure may be sufficient to cause sand boils and piping.

52. The top stratum landward of the levee can be classified into the three following categories as regards thickness: (a) no significant top stratum; (b) top stratum of insufficient thickness to withstand the hydrostatic pressures that tend to develop; (c) top stratum of sufficient thickness to withstand any hydrostatic pressure that may develop at the design flood stage.

53. Where the top stratum landward of a levee is very thin, relatively little hydrostatic head can develop but seepage may be quite heavy. If soil conditions are uniform the intensity of seepage and hydrostatic pressure immediately below the surface will be uniform along the levee. Such seepage is probably not dangerous from the standpoint of subsurface erosion provided the levee has sufficient base width for the possible head on the levee. Where there is no top stratum, the hydrostatic pressure in the upper part of the sand stratum at or landward of the levee toe is usually low. However, natural stratification in the upper part of the sand may cause development of some artesian pressure and concentration of seepage flow in the form of small and probably insignificant sand boils.

54. The potentially dangerous underseepage condition most frequently encountered along the Lower Mississippi River levees is case b. In this case the resistance to seepage flow through the top stratum is so great in comparison to the low resistance to seepage flow through the substratum sands that appreciable artesian pressures are built up beneath the top stratum landward of the levee toe. These artesian pressures above the ground surface at the levee toe during high water generally range from 25 to 75 per cent of the net head on the levee and may extend appreciable distances landward of the levee. In some cases the top stratum may rupture, frequently without warning, where the quantity of seepage may have been very small before the top stratum suddenly ruptured.

55. The third condition presents no underseepage problem except at localized spots where the top stratum may not be entirely continuous or where channels and drainage ditches have been excavated immediately

landward of the levee toe. Where ditches are present, critical substratum pressures may rupture the bottom of the ditch. Resulting sand boils may become quite active because the thick blanket on each side of the ditch will cause subsurface seepage to concentrate in the bottom of the ditch.

56. The amount of underseepage and uplift hydrostatic pressure that may develop landward of a levee is known to be related to the river stage, location of seepage entrance, extent, thickness, and perviousness of the landside top stratum, underground storage, and geological features. (The effect of these foundation characteristics on underseepage and uplift pressure is discussed subsequently.) Other factors contributing to the activity of sand boils caused by seepage and hydrostatic pressure are the increase of river stage above that required to start piping, the degree of seepage concentration, the velocity of flow emerging from the boils, and the existence of a stratum of fine cohesionless soil readily susceptible to piping.

57. Underground storage has a significant effect on underseepage and excess hydrostatic pressures during relatively low high waters or high waters of short duration. If the ground water table is low at the onset of a high water, drainage into subsurface storage landward of the levee will reduce hydrostatic pressures and seepage rising to the surface. However, if the ground water table is high or the flood is of long duration, this factor will have little effect on substratum hydrostatic pressures. In general, piezometric data presented in Part IV indicate that the ground-water storage landward of Lower Mississippi River levees will be filled by the time a project flood stage develops.

Relation of Sand Boils and Piping to Levee Crevasses

58. The levee system along the Lower Mississippi River suffered about 60 major crevasses in the period between 1890 and 1927. The cause of six of these crevasses was reported as sand boils.[39] It is possible that other crevasses, causes of which were listed as "blowouts" or "unknown," may have been caused or significantly influenced by underseepage or piping. During the high water of 1929, piping removed

sufficient foundation material beneath a levee near Greenville, Miss., to cause the levee to settle several feet. The levee was not crevassed because when it settled, the channel through which piping was occurring was cut off, and there was sufficient freeboard on the levee so that it was not overtopped after settling. The incident occurred during the night and was not discovered until the next morning. This incident shows that subterranean erosion, if of sufficient magnitude, may result in gradual or sudden subsidence of the levee to the extent that it could be overtopped.

59. Other crevasses might have occurred as a result of underseepage had not proper efforts been made. Accounts of heavy underseepage and sand boils along the Mississippi River above Vicksburg, and efforts to control them during the 1927 flood, are given in reference 8; detailed reports of underseepage along the Lower Mississippi River levees during the 1937 high water are given in reference 39.

60. Whether a levee would be crevassed as a result of substratum pressures exceeding the critical gradient and concentrated seepage in the form of sand boils or piping, without corrective measures, is practically impossible to predict. However, active sand boils and piping are a potential hazard to the safety of a levee. In general, continuous "pipes" rarely form under a levee but their partial formation can result in progressive collapse of the soil and accelerated erosion which may ultimately undermine the levee. Piping channels revealed by a test pit in a sand boil area at Trotters 54 after the 1937 high water are shown in fig. 10.

Analysis of Seepage Flow and Substratum Pressure

Factors and assumptions in seepage analyses

61. The amount of seepage that will pass beneath a levee and the artesian pressure that will or can develop landward of a levee during a sustained high water are related to and can be estimated from a knowledge of the following factors and characteristics of the foundation. The

Fig. 10. Sand boil and piping channels in a cohesive top stratum, Trotters 54, Miss.

nomenclature used in this section of the report and in subsequent analyses is illustrated in figs. 11 and 18 and is defined in the Notations that precede the text. Definitions of some of the symbols shown on fig. 11 and/or discussed in subsequent paragraphs are given here for convenience.

H , net head on levee.

L_1 , distance from riverside levee toe to river.

L_2 , base width of levee and berm.

L_3 , length of foundation and top stratum landward of the levee toe.

M, slope of hydraulic grade line, at mid-depth of pervious substratum, beneath levee.

z_R and k_{bR} and z_L and k_{bL} , effective thickness and permeability of top stratum riverward and landward of the levee, respectively.

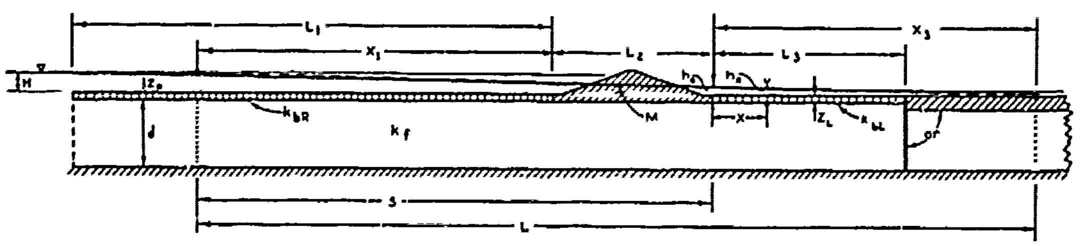


Fig. 11. Generalized cross section of levee foundation and symbols for seepage analysis

d , effective thickness, and k_f , permeability of pervious substratum.

s , distance from landside levee toe to "effective" source of seepage entry into the pervious substratum.

x_3 , distance from landside toe of levee or berm to "effective" seepage exit.

i_c , critical gradient for top stratum landward of the levee.

62. Other factors that influence computation of seepage flow and substratum pressure, but do not lend themselves to theoretical analysis, are stratification of the foundation, lenticular deposits of silts and clays within the foundation, nonuniformity of the top stratum, and riverside or landside borrow pits.

63. Before any seepage analysis by means of theoretical formulas is possible, it is necessary to make certain simplifying assumptions and to generalize the foundation into a pervious sand stratum with a specific thickness and permeability and a semipervious top stratum with a uniform thickness and permeability. (However, the thickness and permeability of the top stratum may be different riverward and landward of the levee.) Seepage may enter the pervious stratum either at the river bank, through riverside borrow pits, and/or through the semipervious top stratum riverside of the levee. Seepage through the pervious substratum is assumed to be horizontal. Flow through the top stratum, or bottom of borrow pits, is assumed to be vertical. The levee, impervious or thick berms, and the portion of the top stratum immediately beneath them, are assumed to be impervious. In most of the theoretical formulas used in this report it is assumed that the ground-water storage landward of the levee is essentially filled and that seepage through the top stratum and in the pervious sands is laminar. (However, seepage beneath a levee can be computed from certain formulas even though flow through the top stratum landward of the levee is no longer laminar.)

Determination of factors
<u>involved in seepage analyses</u>

64. Some of the factors involved in seepage analyses may be determined or estimated by different methods, some more accurate than

others. Methods of determining the necessary factors may include the use of surveys, field explorations, laboratory tests, field pumping tests, and piezometer systems. Methods of arriving at numerical values of these factors, which are subsequently used in Part IV in the analysis of 16 selected sites, are discussed in the following paragraphs.

65. Net head H . The net head on a levee is the height of the flood stage above the tailwater or average low ground surface landward of the levee. For design purposes, H is usually determined from the project flood stage but is sometimes computed on the basis of the net grade of the levee rather than the project flow line.

66. Distance from RS levee toe to river bank L_1 . The distance L_1 can usually be obtained from maps.

67. Base width of levee and berm L_2 . The factor, L_2, can be determined from known dimensions of the levee or by direct measurement.

68. Length of top stratum landward of levee toe L_3 . Often changes in geology and topography will limit the emergence of seepage to a definite area. For example, a wide clay-filled channel immediately landward of the levee may largely prevent the emergence of seepage beyond its near edge. If the ground surface should rise sharply some distance landside of the levee, seepage that comes to the surface must do so between the levee toe and the rise in ground. If such a blocked exit exists, it must be taken into account in seepage analyses. If the distance L_3 to the blocked exit is greater than 1.5 times the "effective" seepage exit length x_3 , the presence of the block may be ignored as its effect on substratum pressures landward of the levee will be less than 10%. The distance to such a block in the landward seepage pattern L_3 can be ascertained from field reconnaissance, geological studies, aerial mosaics, borings, and/or topographic maps.

69. Slope of hydraulic grade line beneath levee M . The slope of the hydraulic grade line in the pervious substratum beneath a levee can best be determined from readings of piezometers located beneath the levee where the seepage flow lines are essentially horizontal and the equipotential lines vertical. M can be determined from piezometer readings obtained during high water and the relation

$$M = \frac{\Delta h}{\ell} \qquad (1)$$

where Δh is difference in piezometer readings, and ℓ is horizontal distance between piezometers. However, this formula is not valid until artesian flow conditions have developed beneath the levee.

70. The tips of piezometers should always be installed in clean sand. If a piezometer at the toe of the levee or berm is to be used for determination of M, s, or x_3, the tip should be located at about the middle of the pervious substratum so as to measure the average head in the pervious aquifer. If the tip of a piezometer at the toe of the levee is at or near the top of the pervious substratum, a correction must be added to the reading to obtain the hydrostatic head at the middle of the aquifer for determination of M, s, or x_3 (see paragraph 132).

71. The hydraulic grade line as determined in the field from piezometer readings is the most reliable method for determining M and the effective seepage entrance and exit. M is also of use in computing the quantity of seepage, Q_s, passing beneath the levee from the formula

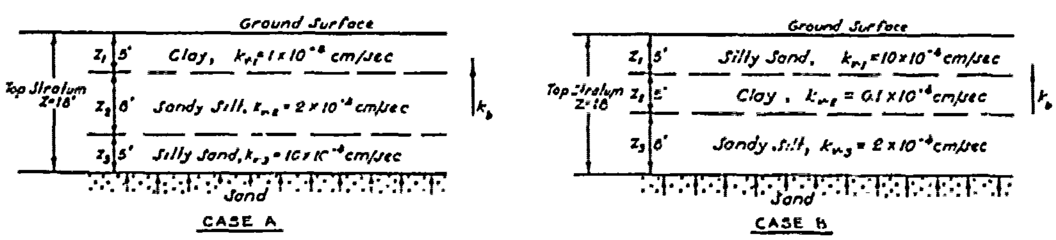
$$Q_s = M\, k_f d \text{ per unit length of levee per unit time .} \qquad (2)$$

72. <u>Effective thickness z_b and permeability k_b of top stratum.</u> The thickness of the top stratum both riverward and landward of a levee is of paramount importance in a seepage analysis. It is usually determined by means of auger borings with samples taken at 3- to 5-ft intervals and at changes in strata. A few undisturbed samples of typical top stratum soils are sometimes taken for permeability tests. The spacing of top stratum or blanket borings depends on the potential severity of the underseepage problem, the surface geology of the area, and variations in characteristics and thickness of the top stratum. The borings should be laid out along the landside toe of the levee or berm so as to sample the basic geological features, with intermediate borings for check purposes. Also sufficient borings should be made to delineate the thickness and extent of any geological features as far as 500 ft landward of the

levee toe that may significantly affect the seepage analysis. The thickness of top stratum remaining in the bottom of manmade ditches landward of a levee should be determined. Good aerial mosaics of the area to a scale of 1 in. = 400 to 1000 ft are indispensable for making a proper layout of blanket borings. The final layout should also be checked in the field before the borings are started. A minimum average spacing of 500 ft is necessary for borings along the toe; however, where there is likely to be an underseepage problem, spacing of 100 to 250 ft is frequently required to delineate the thickness and characteristics of the top stratum.

73. Characteristics of the riverward top stratum are usually estimated from a few riverside borings and by extrapolation of borings made on the landside, together with a study of aerial mosaics. Where the top stratum riverward of the levee has been reduced in thickness in riverside borrow pits, the thickness of the remaining blanket, if any, should be determined by shallow borings, or estimated.

74. As discussed in Part II, the top stratum is seldom composed of one type soil. Instead, it usually consists of several layers of different soils. If the in-situ vertical permeability of each soil is known, it is possible to transform the various layers into a single stratum of a certain thickness and effective permeability as illustrated in fig. 12. Variations in the soil profile even over short reaches, and difficulties in determining the in-situ permeability of blanket soils preclude a precise determination of a generalized top stratum with a specific thickness



(A) $$k_b = \frac{z}{\frac{z_1}{k_{v\text{-}1}} + \frac{z_2}{k_{v\text{-}2}} + \frac{z_3}{k_{v\text{-}3}}} = \frac{18}{\frac{5}{1} + \frac{8}{2} + \frac{5}{10}} \times 10^{-4} = 1.9 \times 10^{-4} \text{ cm/sec}$$

(B) $$k_b = \frac{18}{\frac{5}{10} + \frac{5}{0.1} + \frac{8}{2}} \times 10^{-4} = 0.33 \times 10^{-4} \text{ cm/sec}$$

Fig. 12. Computation of vertical permeability of top stratum

and permeability. However, if good judgement is exercised in the selection of these values, reasonably accurate seepage analyses can be made, and seepage control measures can be designed satisfactorily.

75. The in-situ permeability of clay strata in the top blanket is related to the thickness of clay, whether or not it is at or near the surface or covered by natural levee deposits, and to a large extent to the presence of root holes, shrinkage cracks, minute fissures, and burrows of crayfish and small animals. Flow up through relatively thin (<5 ft) clay strata near the surface is generally through these channels rather than through the pores of the soil. Tests on small samples of clay in the laboratory measure the permeability of the pores in the soil mass and are not usually indicative of the permeability of clay strata at or near the surface. A clay top stratum landward of a levee is considered more pervious during high water than one on the riverside because of the flushing action of seepage rising through small channels to the surface on the landside whereas on the riverside such small channels tend to silt up.

76. The in-situ vertical permeability of semipervious soils such as silty sand, sandy silt, and silt can be determined reasonably accurately from laboratory tests on undisturbed samples as the flow through these soils is usually laminar unless sand boils have developed in the area.

77. In the computation of seepage flow and landward substratum pressures, the effective vertical permeability k_b is the average permeability of the strata that make up the top stratum as long as the seepage flow is laminar. The effective permeability can be estimated from the following formula

$$k_b = \frac{z}{\frac{z_1}{k_{v-1}} + \frac{z_2}{k_{v-2}} + \frac{z_3}{k_{v-3}}} \quad (3)$$

Examples of computations to determine k_b are shown in fig. 12.

78. The top stratum may also be generalized into a blanket of uniform vertical permeability with a specific effective thickness by transforming the actual thickness of various strata to another thickness

with a certain permeability, as illustrated in the examples below and in fig. 13.

Strata	Actual z_n	Actual $k_{v-n} \times 10^{-4}$ cm/sec	Thickness Transformation Factor $\frac{k_b}{k_{v-n}}$	Transformed, z_{b-n} for $k_b = 1 \times 10^{-4}$ cm/sec
Clay	5 ft	1	1.0	5.0
Sandy silt	8 ft	2	1/2	4.0
Silty sand	5 ft	10	1/10	0.5
Top stratum thickness z =	18 ft	--	----	Transformed z_b = 9.5

This procedure for generalizing the top strata into a blanket of uniform vertical characteristics for seepage analyses is the one subsequently used in the analysis of the data from the piezometer sites, and in the investigation of underseepage along the levees in the St. Louis District.[54] Transformation factors used in the St. Louis District studies, which were based to a considerable extent on data from the 16 piezometer sites, are given in table 1. The basis for the transformation factors given in table 1 is subsequently discussed in Part IV.

79. Of equal importance is the thickness of the landside top stratum to be used in determination of the allowable pressure beneath the top stratum for design of seepage control measures. This effective thickness z_t may or may not be the same as that used in seepage analyses. Where the most impervious soil is at the ground surface, the effective thickness computed by transforming the various components of the top stratum to one thickness of a single permeability, as illustrated in fig. 12, is also satisfactory for computation of the maximum allowable, or critical, substratum pressure (see Case I, fig. 13). If semipervious material overlies less pervious material as in Case II, fig. 13, the effective thickness z_t as regards critical uplift is the total thickness above the bottom of the less pervious strata. This thickness may be considerably greater than the transformed thickness z_b for computation of "effective" seepage exit length (see fig. 13).

80. Where the top stratum is comprised of alternating strata of

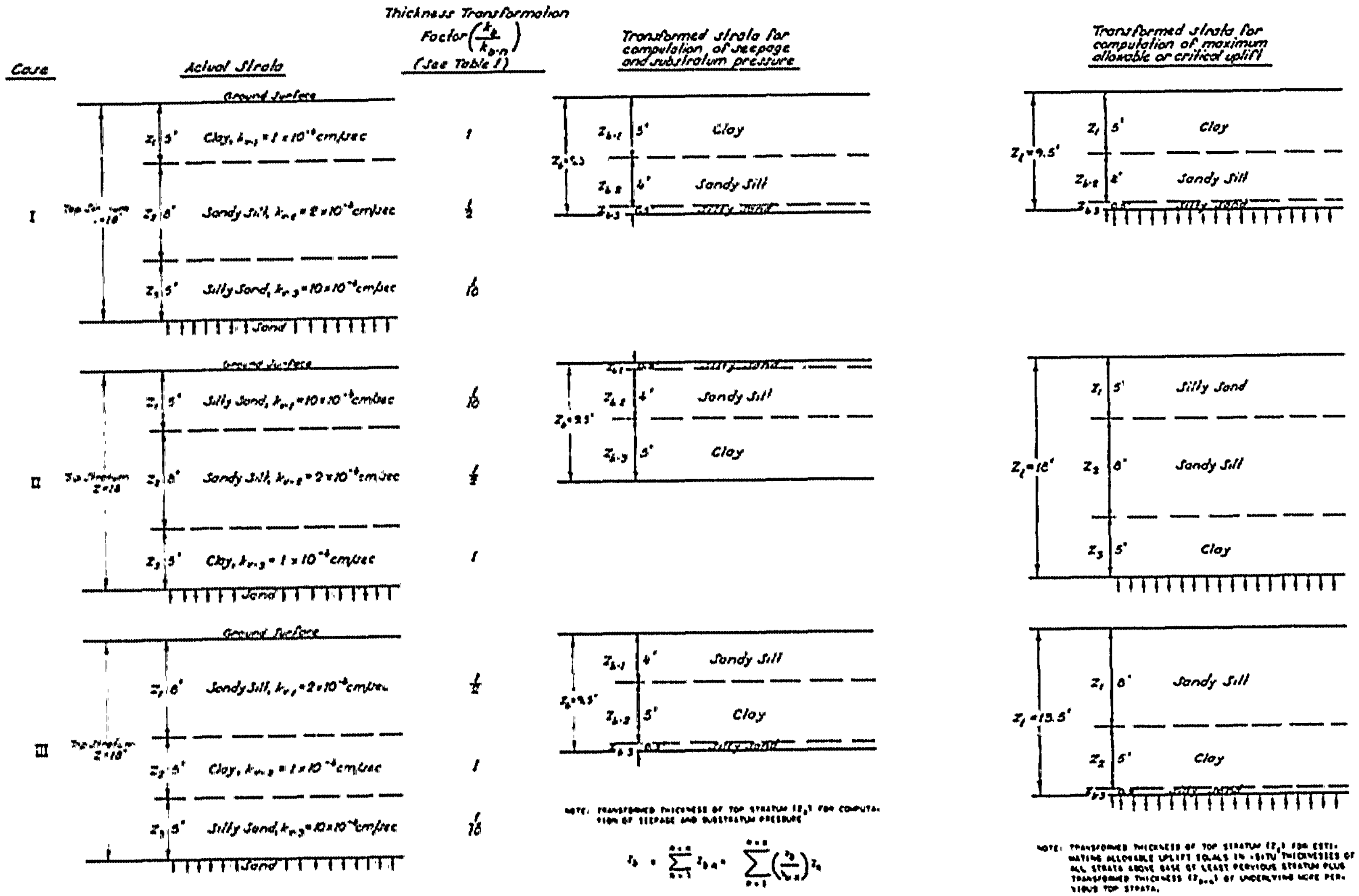


Fig. 13. Transformation of top strata

Table 1

Thickness Transformation Factors for Top Strata

Soil Type		
LMVD	Unified Soil Class. System	Transformation Factor
	Clay less than 5 ft in Thickness	
Clay	Fat clay (CH)	1
Silty clay	Lean clay (CL)	1
Clay silt	Silt (ML)	1
Sandy silt	Silt, sandy (ML)	3/4 to 1
Silty sand	Silty sand (SM)	1/5 if $z < 10$ ft; 0 if $z > 10$ ft
Very fine sand	Fine sand	0
Alternating clay and silt strata with depth		1
	Clay more than 5 ft in Thickness	
Clay	Fat clay (CH)	1
Silty clay	Lean clay (CL)	1
Clay silt	Silt (ML)	1/2
Sandy silt	Silt, sandy (ML)	1/4 to 1/2 if $z < 10$ ft; 0 if $z > 10$ ft
Silty sand	Silty sand (SM)	1/10 if $z < 10$ ft; 0 if $z > 10$ ft
Very fine sand	Fine sand	0
Alternating clay and silt strata with depth		1

varying perviousness as in Case III, fig. 13, the effective thickness for computing effective exit length is the transformed thickness z_b. The thickness for critical uplift considerations z_t is usually based on the total thickness of the various strata overlying the base of the least pervious stratum added to the transformed thickness z_{b-n} of underlying more pervious top strata.

81. The effective vertical permeability k_{bL} of the top stratum landward of a levee can also be computed from observed hydrostatic heads beneath the landside top stratum, together with seepage measurements, and the following formula:

$$k_{bL} = \frac{Q_A \; z_{bL}}{h_x \; A} \; . \tag{4}$$

82. The permeability of the top blanket k_{bL} can also be computed from known characteristics of the pervious foundation and the effective seepage exit x_3 as determined from the hydraulic grade line in the pervious foundation beneath the levee using the following formulas:

Where $L_3 = \infty$,

$$k_{bL} = \frac{z_{bL} \; k_f \; d}{x_3^2} \; . \tag{5}$$

Where L_3 = a finite distance ,

$$c \tanh cL_3 = \frac{1}{x_3} \tag{6}$$

where

$$c = \sqrt{\frac{k_{bL}}{k_f \; z_{bL} \; d}} \; . \tag{7}$$

In formulas 6 and 7 k_{bL} has to be determined by trial and error. (If $\frac{k_f}{k_{bL}}$ is less than 100 to 500, the value of k_{bL} computed from x_3 will be slightly low, because of the loss in head up through the aquifer at freely seeping sites.)

83. In making a final determination of the effective thickness and permeability of the top stratum it is necessary to consider the characteristics of the top stratum landward of the levee toe for a distance of at least 200 or 300 ft. It is also necessary to make certain averaging assumptions where geological and soil conditions are reasonably similar. Thin or critical spots should be given considerable weight in arriving at such averages.

84. In seepage analyses the various layers of soil making up the top stratum generally are transformed to a single blanket with a permeability equal to that of the most impervious stratum. This procedure was

used in arriving at the top stratum thicknesses shown on the surface geology maps for the 16 piezometer sites, and in analyses of these sites.

85. Where borrow pits, ditches, or channels exist within 200 or 300 ft of the landside levee toe, the thickness of top stratum used in computing seepage flows and substratum pressures should be based on the thickness of the top stratum adjacent to the ditch, unless the ditch or borrow pit is very wide. The allowable critical substratum pressure must be computed for both the thickness of the top stratum at the toe of the levee and in the bottom of the ditch. The more critical condition or pressure will depend on the depth and location of the ditch.

86. Effective thickness d and permeability k_f of pervious substratum. The thickness of the pervious substratum usually is defined as the thickness of the principal seepage-carrying sand stratum below the top stratum and above the bottom of the entrenched valley (see fig. 11). It may be determined by means of deep borings or a combination of shallow borings and seismic or electrical resistivity surveys. (The thickness of very fine sand strata of low permeability that frequently exist between the top stratum and the principal sand aquifer is usually ignored in seepage and pressure computations.) The thickness of individual sand strata within the principal sand aquifer must be obtained by individual deep borings over the area.

87. The average horizontal permeability k_f of the pervious substratum is best determined by means of a field pumping test on a well that fully penetrates the pervious aquifer. Such tests should include determination of well flow for two or three different drawdowns, the drawdown curve out from the test well and the radius of influence. Where feasible, the flow in the well should be measured by means of a well flow meter[52] at major changes in sand strata. For soil conditions usually existing in the Lower Mississippi River Valley, the permeability of the pervious substratum is best computed from the following formula for artesian flow. (The top stratum and upper fine sands are usually so much less pervious than the underlying deeper sands that they in effect create an upper impervious boundary. The lower impervious boundary is formed by the deep Tertiary materials.)

$$Q_w = \frac{2\pi\ k_f d\ (h_1 - h_2)}{2.30\ \log_{10} \frac{r_2}{r_1}} \tag{8}$$

or

$$k_f = \frac{2.30\ Q_w\ \log_{10} \frac{r_2}{r_1}}{2\pi\ d\ (h_1 - h_2)} \tag{8a}$$

88. Where the permeabilities of different sand strata in the pervious substratum are significantly different, as illustrated in fig. 14, the permeability of individual sand strata can be computed from the difference in the well flows in the screen at the boundaries of the sand stratum being tested using the formula on the following page.

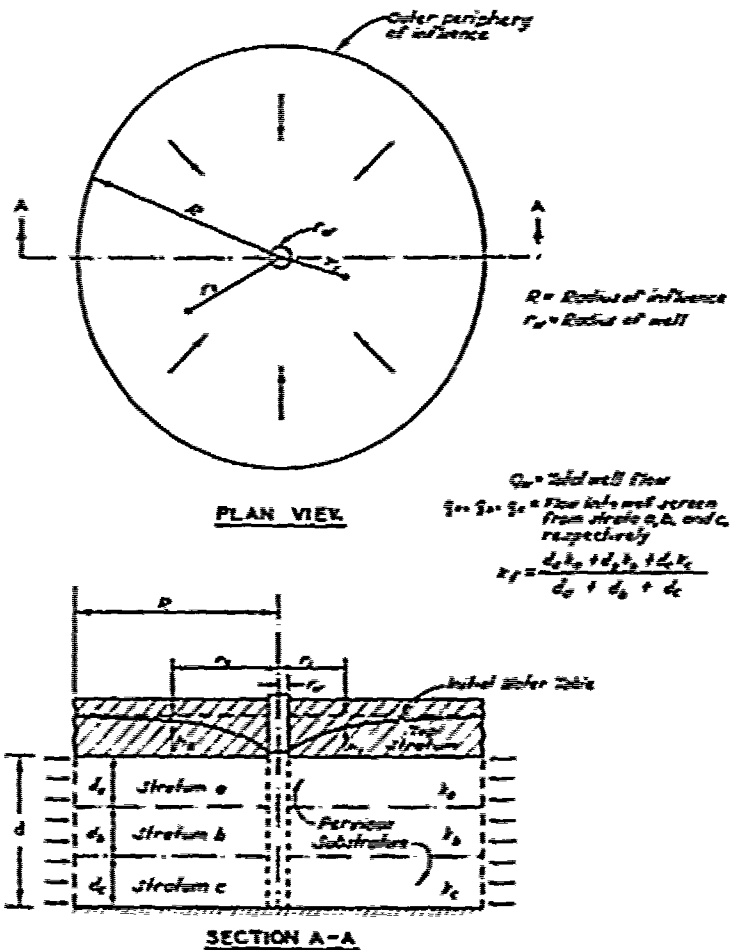


Fig. 14. Artesion flow to a test well

$$k_{f(a)} = \frac{2.30\, q_a \log_{10} \frac{r_2}{r_1}}{2\pi d_a\, (h_1 - h_2)} . \tag{8b}$$

The method used in the field pumping tests made in the St. Louis and New Orleans Districts to determine both the average permeability and the permeability of individual sand strata of the pervious aquifer and the results of these tests are given in Appendix C. Where only grain size or laboratory permeability test data are available for different sand strata, the average k_f can be computed from the following formula:

$$k_f = \frac{d_a k_a + d_b k_b + d_c k_c + d_n k_n}{d_a + d_b + d_c + d_n} . \tag{9}$$

89. The average horizontal permeability can also be determined from pumping tests on partially penetrating wells by using the straight line portion of the drawdown curve (with r plotted to a semilog scale) some distance from the well where flow lines to the well are essentially horizontal and are not affected by the curved pattern of flow in the vicinity of the well, and equation 8a. If the pervious stratum is homogeneous and $k_v = k_H$, the average permeability can be approximated from equation 8a, modified as follows:

$$k_f = \frac{2.30\, Q_w \log_{10} \frac{R}{r_w}}{2\pi\, d\, h\, G} \tag{8c}$$

where

G = ratio of flow of a partially penetrating to a fully penetrating well computed from Muskat's formula as determined from fig. 15.

90. When it is not possible to determine k_f from pumping tests, it may be estimated from mechanical analyses or laboratory permeability tests on samples of sand taken by means of a Shelby tube or split spoon sampler in a "mudded" hole, or a piston-type bailer. If no gravel is present, nearly undisturbed samples of sand can be obtained with a Shelby tube sampler in a "mudded" hole. The next best sand samples are obtained with a piston-type bailer. Samples taken with either a Shelby tube or

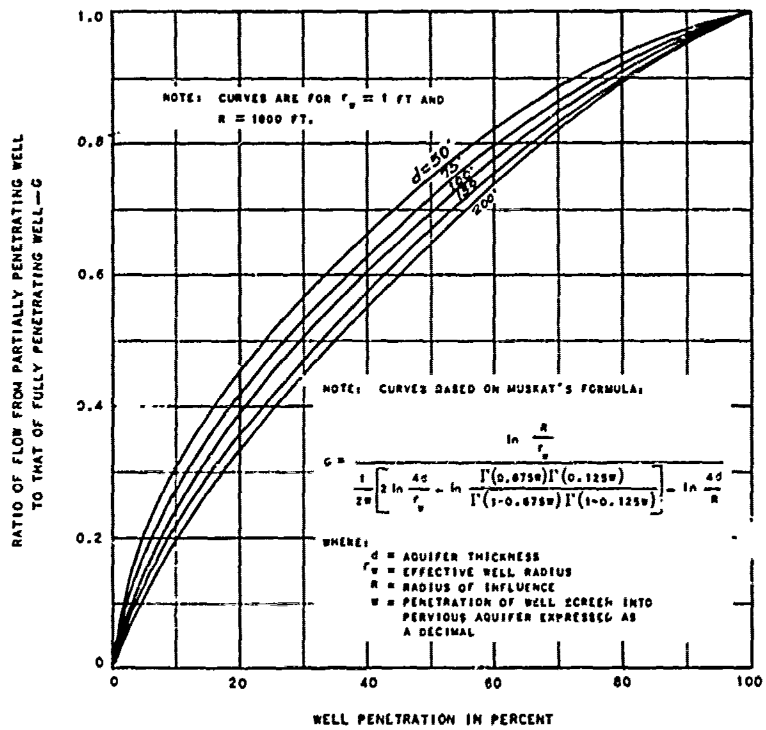


Fig. 15. Relation between flow from a partially penetrating artesian well in a homogeneous foundation and from a fully penetrating well

split spoon sampler in holes bored with drilling mud must be thoroughly and carefully washed of all drilling mud before testing. The relation of the effective grain size D_{10} of sand samples taken by bailer samplers at the piezometer sites to the permeability of remolded sand samples as determined in the laboratory k_R, and adjusted to a temperature of 20 C and an assumed void ratio e of 0.60, is shown on plate 244. Since plate 244 was prepared, a relationship has been found[48] between D_{50} and e for sands in the lower alluvial valley, as shown in fig. 16, which permits a more accurate estimation of the in-situ void ratio than assuming $e = 0.60$.

91. The pumping tests described in Appendix C have shown that the actual horizontal permeability of a sand stratum is 1.5 to 4 times greater than the permeability indicated by laboratory tests on remolded samples taken by any of the previously mentioned sampling methods. An approximate empirical relationship between D_{10} and k_H developed from the data in Appendix C is shown in fig. 17. If field pumping tests are not

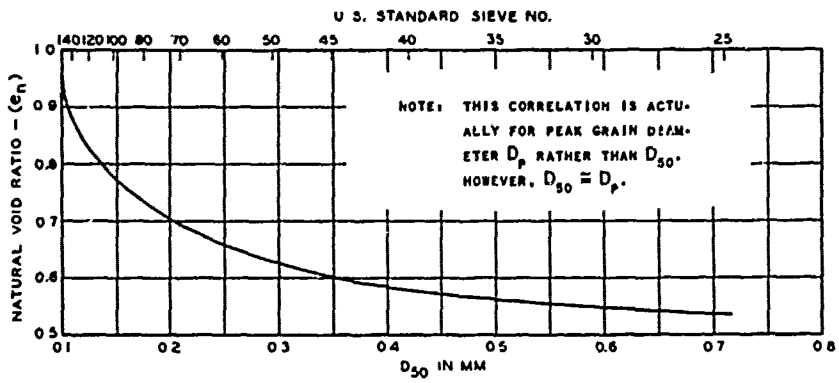


Fig. 16. Grain size D_{50} vs natural void ratio e_n

performed, it is believed that the horizontal permeability of a sand stratum can be estimated more accurately from fig. 17 than from either plate 244 or from laboratory permeability tests on remolded samples.

92. The average horizontal permeability of the pervious strata beneath a levee can also be estimated from the hydraulic grade line beneath the levee and total seepage passing beneath the levee.

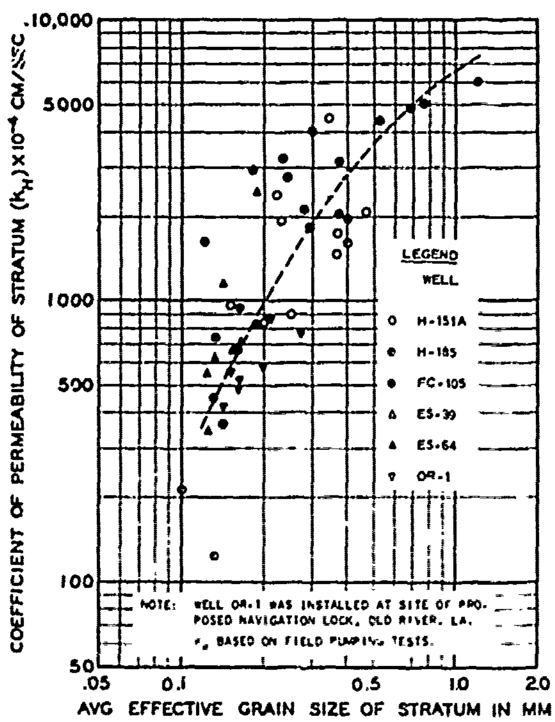


Fig. 17. In-situ horizontal permeability vs effective grain size D_{10}

$$Q_s = k_f M d \qquad (2)$$

or

$$k_f = \frac{Q_s}{M d} \qquad (2a)$$

where the rate of seepage flow per unit length of levee Q_A emerging in area A is measured, the permeability of the pervious substratum can be computed from the formula

$$k_f = \frac{Q_A}{(M - M_A)\ d} . \tag{2b}$$

93. <u>Effective source of seepage entry s .</u> The effective source of seepage entry into the pervious substratum, as illustrated in fig. 11, is defined as that line riverward of the levee where a hypothetical open seepage entry face fully penetrating the pervious aquifer with an impervious blanket between this line and the levee would produce the same flow and hydrostatic pressure beneath and landward of a levee as will occur for the actual conditions riverward of the levee. It may also be defined as that line or point where the hydraulic grade line beneath the levee projected riverward with slope M intersects the river stage.

94. The best and most accurate method for determining the distance from the landside levee toe to the effective source of seepage entry $(s = x_1 + L_2)$ is to project graphically the hydraulic grade line M beneath the levee, as measured by piezometers installed in the pervious substratum beneath the levee, until it intersects the river stage producing the gradient. The value of s can also be determined from piezometric data using the following equations (see fig. 18 for nomenclature):

$$s = \ell_1 + (H - h_1)\frac{(\ell_2 - \ell_1)}{(h_2 - h_1)} . \tag{10}$$

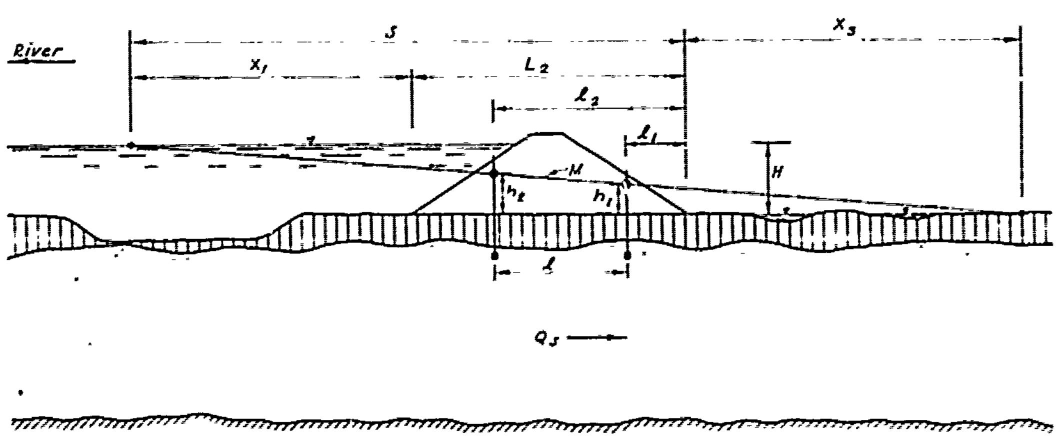


Fig. 18. Nomenclature for determination of s and x_3 from piezometer readings

$$s = \ell_1 + \frac{H - h_1}{M} . \tag{10a}$$

Before equations 10 and 10a are valid, the pervious stratum beneath the levee must be saturated or artesian flow conditions established. These methods of determining s automatically integrate the many rather indeterminate factors that influence the entry of seepage into the pervious substratum, and were used for determining the source of seepage at the 16 piezometer sites discussed in Part IV. Design values of s or x_1 determined from piezometer readings should be based on the highest river stages for which piezometer data are available or extrapolated from plots of s or x_1 vs river stage as illustrated in Part IV.

95. If the distance to the river $L_1 + L_2$ (fig. 11) is known and there are no riverside borrow pits, the distance to the effective source of seepage entry $s = x_1 + L_2$ can be estimated from the following formula:

$$x_1 = \frac{\tanh c\, L_1}{c} \tag{11}$$

where

$$c = \sqrt{\frac{k_{bR}}{k_f\, z_{bR}\, d}} . \tag{11a}$$

Where a block, or wide, thick deposit of clay exists a certain distance riverward of a levee so as to prevent any entrance of seepage into the foundation beyond that point, the distance to the effective source of seepage $s = x_1 + L_2$ can be estimated from the following equation for x_1 :

$$x_1 = \frac{1}{c \tanh cL_1} \tag{12}$$

where

L_1 = distance from riverside toe of levee to the block.

Where two guide levees parallel a tributary stream or a floodway channel, and seepage into the foundation is divided and the bottom of the tributary

stream or channel does not expose foundation sands, x_1 can also be computed from equation 12 wherein L_1 equals half the distance between the riverside toes of the levees. (The term c in equation 12 would be computed from equation 11a.) The above described condition is typical of flank or guide levees along tributaries of the Mississippi River between Alton and Gale, Ill. (St. Louis District).[54]

96. Where riverside borrow pits have been dug causing most of the impervious stratum over a considerable area to be removed, and the pits become the primary entrance for seepage, s may be estimated from equation 12 where z_{bR} and k_{bR} are values for the top stratum remaining in the bottom of the borrow pit or as illustrated in fig. 19. If the flow downward through the bottom of the borrow pit (stratum 1) and the upper strata of silty sand (stratum 2) as shown in fig. 19 is assumed to be vertical, the thickness of these strata, z_1 and z_2, may be converted into equivalent lengths, s'_1 and s'_2, of foundation with a depth of d and permeability of k_f. For the conditions shown in fig. 19, $Q_s = Q_1 = Q_2$. Replacing top strata 1 and 2 in the borrow pit with an equivalent length of foundation:

$$s'_1 = \frac{k_f}{k_{v-1}} \times \frac{d}{d_1} \times z_1 = \frac{500 \times 10^{-4}}{0.5 \times 10^{-4}} \times \frac{100}{150} \times 2 = 1333 \text{ ft}$$

$$s'_2 = \frac{k_f}{k_{v-2}} \times \frac{d}{d_2} \times z_2 = \frac{500 \times 10^{-4}}{10 \times 10^{-4}} \times \frac{100}{150} \times 10 - 333 \text{ ft} .$$

The effective distance s from the landside toe of the levee to the effective seepage entrance assuming an impervious top stratum and all horizontal flow to be carried by the more pervious foundation stratum is

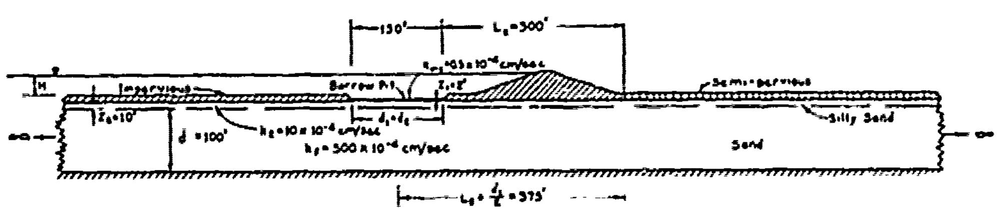


Fig. 19. Estimation of effective source of seepage where riverside borrow pits exist

$$s = L_2 + \frac{d_1}{2} + s'_1 + s'_2 \tag{13}$$

$$s = 300 + \frac{150}{2} + 1333 + 333 = 2040 \text{ ft} .$$

97. Distance from landside levee toe to effective seepage exit x_3. The effective seepage exit is defined as that line or point landward of the levee where a hypothetical open drainage face and an impervious blanket between this point and the levee toe would result in the same hydrostatic pressure at the levee toe and would cause the same amount of seepage to pass beneath the levee as would actually occur for existing conditions. The distance x_3 to this point is the intersection of an extension of the hydraulic grade line beneath the levee with the ground surface or tailwater (see fig. 11). The best way to determine x_3 is by means of piezometers installed in the pervious substratum beneath a levee using the following expression (see fig. 18 for nomenclature):

$$x_3 = h_1 \frac{(\ell_2 - \ell_1)}{(h_2 - h_1)} - \ell_1 \tag{14}$$

or

$$x_3 = \frac{h_1}{M} - \ell_1 . \tag{14a}$$

As x_3 may vary with river stages as seepage develops, x_3 should be plotted vs river stage, and the estimated maximum value obtained from a curve of best fit should be used for seepage analyses.

98. The distance to the effective seepage exit can also be estimated from blanket formulas:

where $L_3 = \infty$

$$x_3 = \frac{1}{c} = \sqrt{\frac{k_f \, z_{bL} \, d}{k_{bL}}} \tag{15}$$

where L_3 = finite distance to a block

$$x_3 = \frac{1}{c \tanh c \, L_3} \tag{16}$$

where L_3 = finite distance to an open exit

$$x_3 = \frac{\tanh c L_3}{c} . \tag{17}$$

99. The relationship between c and the effective seepage exit length x_3 where the semipervious top stratum is infinite in landward extent (case 5, fig. 23) has been computed from equation 15 and plotted in fig. 20 for various values of k_f/k_b assuming $d = 100$ ft . The x_3 corresponding to values of d other than 100 ft can be computed from the equation

$$x_3 = 0.1 \sqrt{d}\, x_{3(d = 100)} . \tag{18}$$

100. Where the landside top stratum has a finite length L_3 with either a block or open seepage exit at L_3 (see fig. 23, cases 6 and 7, respectively), the effective exit length can be computed either from equation 16 or 17 or by multiplying $x_{3(L_3 = \infty)}$ by the factor shown in fig. 21. Figs. 20 and 21 can also be used to evaluate the effective length of riverside blanket x_1 by using L_1 , z_{bR} , and k_{bR} for L_3 , z_{bL} , and k_{bL} , respectively.

101. Combinations of s and x_3 . Various combinations of s and x_3 for use in the design of underseepage control measures can be estimated from the reading of a single piezometer at a fairly high river stage, by determining the distances to the source of seepage entry and the effective seepage exit required to create an h_o equal to the observed head at the toe of the levee. The distance to the source of seepage can be estimated from reaches where piezometers have been installed perpendicular to the levee and where riverside soil conditions are of a similar nature.

102. Ratio k_f/k_{bL} . The ratio k_f/k_{bL} can be computed from equations 5-7 and values of x_3 determined from the hydraulic gradient beneath the levee without knowing either k_f or k_{bL} .

103. Critical gradient i_c . The critical gradient required to

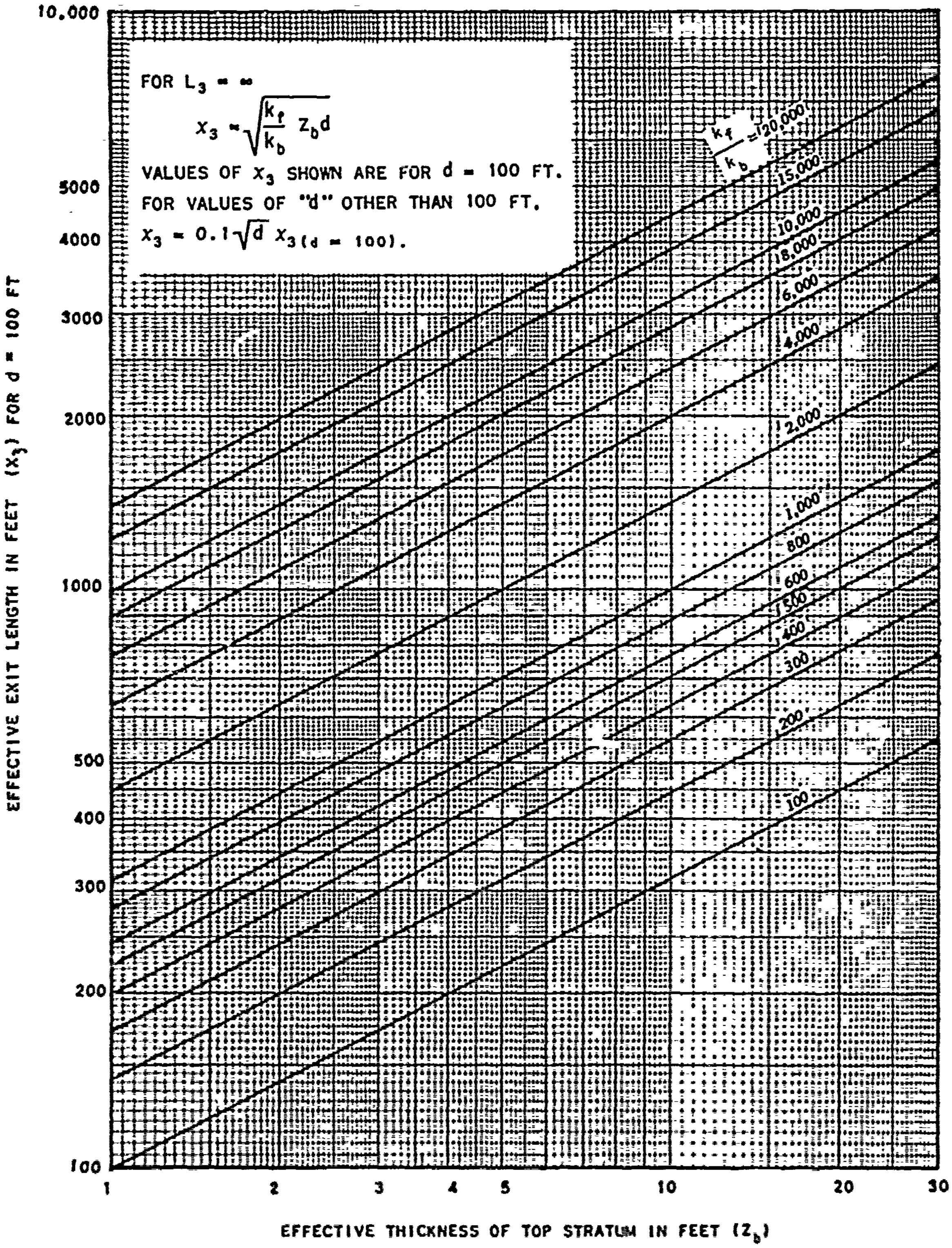


Fig. 20. Effective seepage exit length for $L_3 = \infty$ and d = 100 ft

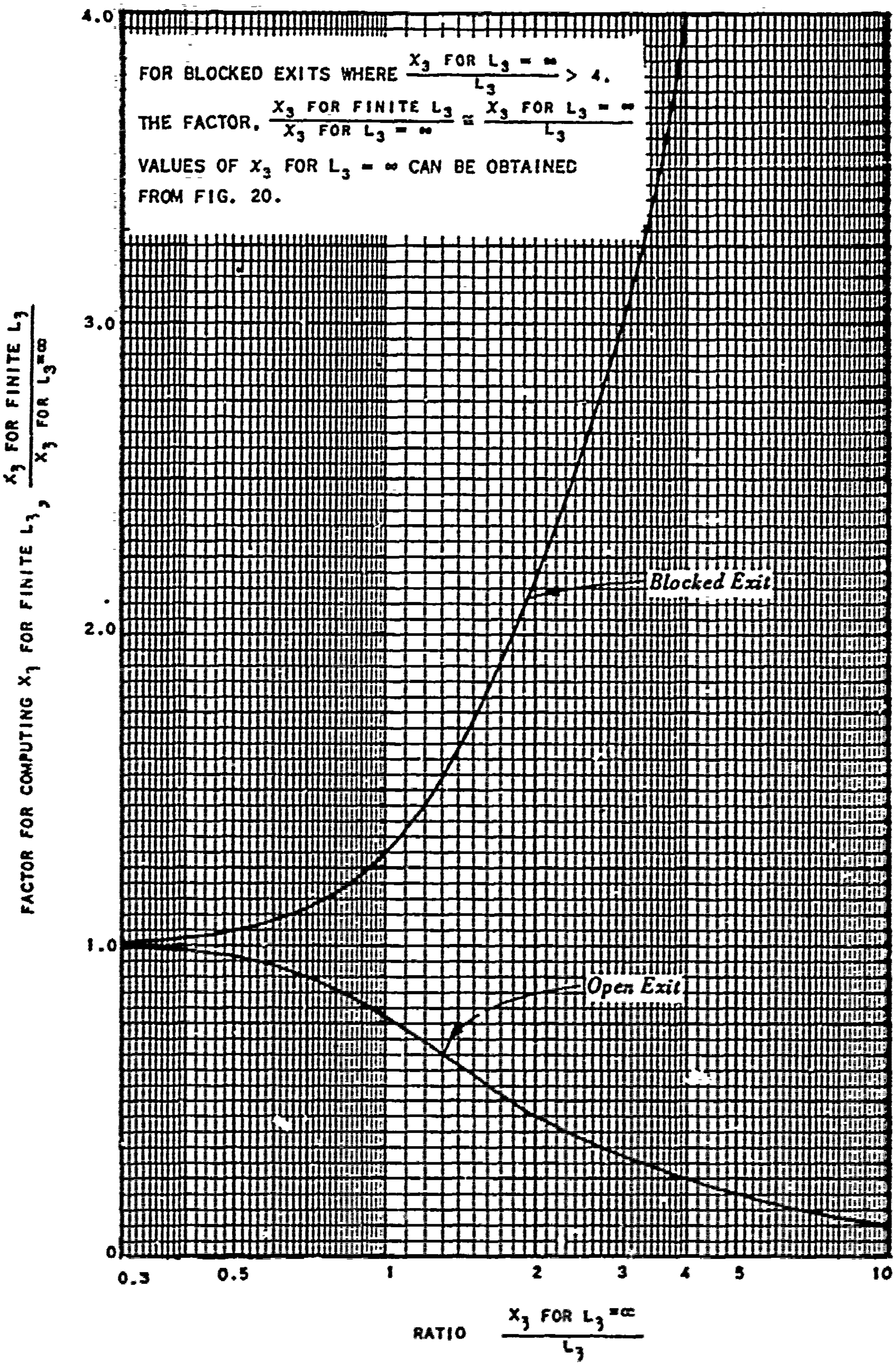


Fig. 21. Ratio between x_3 for blocked or open exits and x_3 for $L_3 = \infty$

cause sand boils or heaving or flotation of the top stratum is usually defined as the ratio of the submerged unit weight of the soil γ' comprising the top stratum and the unit weight of water γ_w, or

$$i_c = \frac{\gamma'}{\gamma_w} = (G - 1)\ (1 - n) = \frac{G - 1}{1 + e} \quad . \tag{19}$$

Homogeneous soils have the following approximate theoretical critical gradients:

Soil Type	i_c
Silty sand and silts	0.85
Silty clay and clay	0.80

104. The critical gradient required to cause sand boils in the field can best be determined by measuring the hydrostatic head beneath the top stratum at the time sand boils first appear. In this method i_c is determined from the following formula:

$$i_c = \frac{h_{x(c)}}{z_t} \quad . \tag{20}$$

Critical gradients as measured in the field at some of the piezometer sites are discussed in Parts IV and V.

Formulas for computation of seepage flow and substratum hydrostatic pressures

105. Engineers frequently are required to predict the severity of underseepage at sites where piezometric data and seepage measurements are not available. An estimate of seepage flow and substratum pressures can be made from mathematical formulas, provided soil conditions at the site are known. Formulas for estimating the head landward of and seepage beneath simplified, generalized sections of levees and foundation soils existing in the Lower Mississippi River Valley are given in the following paragraphs. These formulas were also used in analyzing piezometric and seepage data from the sites discussed in Part IV. It is emphasized that the accuracy of the results obtained from the formulas presented is dependent upon the applicability of the formula to the condition being

analyzed, the uniformity of soil conditions, and evaluation of the various factors involved in the computations.

106. For a levee underlain by a pervious foundation, the natural seepage Q_s per unit length of levee can be expressed by the following general formula

$$Q_s = \$ k_f H . \tag{21}$$

This equation is valid provided the assumptions on which Darcy's law is based are met. The mathematical expression for $\$$ depends upon the dimensions of the generalized cross section of the levee and foundation, the characteristics of the top stratum riverward and landward of the levee, and the pervious substratum. Where the hydraulic grade line M beneath the levee is known from piezometer readings, seepage passing beneath the levee can always be determined from the formula

$$Q_s = M k_f d \text{ per unit length of levee per unit time} . \tag{2}$$

107. The excess hydrostatic head h_o beneath the top stratum at the landside toe of the levee is related to the net head on the levee, the dimensions of the levee and foundation, permeability of the foundation, and the character of the top stratum riverward and landward of the levee. The head h_o can be expressed as a function of the net head H as subsequently shown in this report.

108. The head h_x beneath the top stratum at a distance x landward from the landside toe of the levee can be expressed in terms of the net head H and distance x , although it can be more conveniently related to the head h_o at the levee toe. When h_x is expressed in terms of h_o it depends only upon the type and thickness of the blanket and pervious foundation landward of the levee; the ratio h_x/h_o is independent of conditions riverward of the levee. Expressions for the ratio h_x/h_o for various typical conditions are presented subsequently.

109. Expressions for $\$$, h_o/H , and h_x/h_o for typical levee and foundation conditions along the Lower Mississippi River are discussed in the following paragraphs.

110. <u>No top stratum.</u> Where a levee is founded directly on pervious foundation sands and no top stratum exists either riverward or landward of the levee (see fig. 22, case 1), the seepage Q_s per unit length of levee can be determined from equation 21
in which

$$\S = \frac{d}{L_2 + 0.86\, d} \quad (22)$$

The excess hydrostatic head landward of the levee at the top of the sand foundation is zero; thus $h_o = h_x = 0$. The severity of such a condition in nature is governed by the exit gradient and seepage velocity that develop at the landside toe of the levee; these can be estimated from a flow net compatible with the value of $\S$ computed above.

111. Some idea of the safety of a levee against piping where no top stratum exists may also be obtained from comparison of a computed creep ratio and recommended empirical values thereof. The creep ratio can be computed by either of the following formulas proposed by Bligh[4] or Lane,[22] and the answers therefrom compared with the respective minimum values listed in table 2.

$$\text{Bligh's creep ratio, } C = \frac{L_2}{H} \quad (23)$$

$$\text{Lane's "weighted" creep ratio, } C_w = \frac{1/3\, L_2 + \Sigma p}{H} \quad (24)$$

where

Σp = sum of shortest vertical paths of seepage flow around cutoffs beneath a levee.

Table 2

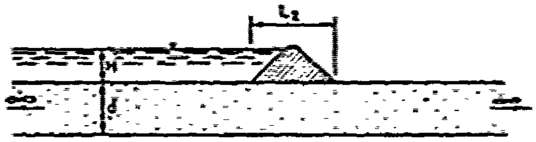
<u>Minimum Creep Ratios</u>

Material	Creep Ratios: Bligh C	Creep Ratios: Lane C_w
Very fine sand or silt	18	8.5
Fine sand	15	7
Medium sand	--	6
Coarse sand	12	5
Fine gravel or sand and gravel	9	4
Coarse gravel including cobbles	4 to 6	3
Boulders with some cobbles and gravel	--	2.5

Basic Equations and Definitions

Seepage per unit length of levee .... $Q_s = f k_f H$
Head beneath top stratum at landside toe of levee .... h_o
Head beneath top stratum at x distance landward from landside toe of levee .... h_x

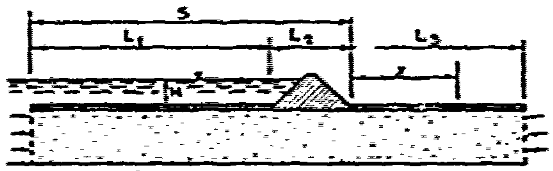
CASE 1



$$f = \frac{d}{L_2 + 0.86d}$$

$\frac{h_o}{H}$ and $h_x = 0$

CASE 2



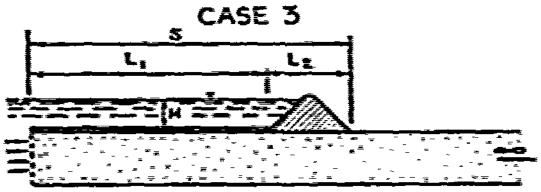
$$f = \frac{d}{L_1 + L_2 + L_3} = \frac{d}{s + L_3}$$

$$\frac{h_o}{H} = \frac{L_3}{s + L_3}$$

For $x \leqq L_3$, $\frac{h_x}{h_o} = \frac{L_3 - x}{L_3}$

For $x \geqq L_3$, $h_x = 0$

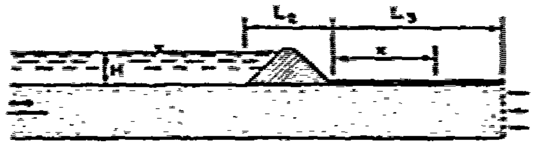
CASE 3



$$f = \frac{d}{L_1 + L_2 + 0.43d} = \frac{d}{s + 0.43d}$$

$\frac{h_o}{H}$ and $h_x = 0$

CASE 4



$$f = \frac{d}{0.43d + L_2 + L_3}$$

$$\frac{h_o}{H} = \frac{L_3}{0.43d + L_2 + L_3}$$

$$\frac{h_x}{h_o} = \frac{L_3 - x}{L_3}$$

Note:
Formulas for f and $\frac{h_o}{H}$ for cases 3 and 4 also are applicable where the indicated impervious top stratum is semi-pervious provided the effective lengths of blanket x_1 and x_3 are substituted in the formulas for L_1 and L_3 respectively

Fig. 22. Formulas for computation of seepage flow and substratum pressures for various top strata

112. <u>Impervious top stratum.</u> A somewhat uncommon condition is that where the top stratum landward of the levee is almost completely impervious. Such a condition is approximated, however, where levees are founded on thick (>15 ft) deposits of clay, or silts with clay strata. For the condition of an impervious landside top stratum little or no seepage occurs through the top stratum and, if the top stratum is either infinite in landward extent or the pervious aquifer is blocked landward of the levee, no seepage occurs beneath the levee (Q and $\$ = 0$) and a hydrostatic head equal to the net head H on the levee develops beneath the landside top stratum. Thus $h_o = h_x = H$.

113. If the top stratum is impervious between the levee and river and has a length L_1, and if the top stratum is impervious from the levee landward to an open seepage exit at a distance L_3 (see fig. 22, case 2), the distance from the landside toe of the levee to the effective seepage entry is $s = (L_1 + L_2)$ and

$$\$ = \frac{d}{L_1 + L_2 + L_3} = \frac{d}{s + L_3} \tag{25}$$

and

$$Q_s = k_f H \frac{d}{s + L_3} . \tag{26}$$

The head h_o beneath the top stratum at the landside levee toe can be computed from the formula

$$h_o = H \left[\frac{L_3}{L_1 + L_2 + L_3} \right] . \tag{27}$$

The head h_x at a distance x landward from the landside levee toe is:

$$\text{for } x < L_3,\ h_x = h_o \left(\frac{L_3 - x}{L_3} \right) \tag{28}$$

$$\text{for } x \geq L_3,\ h_x = 0 . \tag{28a}$$

114. A condition of no top stratum landward of a levee (case 3, fig. 22) is sometimes encountered where an extensive landside borrow pit

has been excavated, resulting in the removal of all top stratum landward of the levee for a considerable distance. The excess head at the top of the sand is zero everywhere landward of the levee and thus the danger from piping for the case in question must be evaluated from the upward gradient obtained from a flow net, or by some empirical method. Seepage beneath the levee can be computed from the shape factor $\$$ given by the formula for case 3 in fig. 22.

115. Where extensive riverside borrow pits have resulted in exposal of foundation sands for a considerable distance riverward of the levee (case 4, fig. 22), the head h_o at the landside toe of the levee and the seepage per unit length of levee can be computed from the formulas given in case 4 in fig. 22. It should be noted that these formulas can be used to estimate the seepage flow beneath the levee and the head h_o for conditions with semipervious instead of impervious landside top stratum, since the effect of a semipervious blanket on Q_s and h_o can be duplicated by an equivalent finite length of impervious blanket.

116. Semipervious top stratum. The condition most commonly encountered is that where a semipervious top stratum overlies the pervious substratum. The formulas that follow are based on the assumption that seepage flow through the top stratum is in a vertical direction and seepage in the pervious substratum is in a horizontal direction. These assumptions are essentially valid wherever the permeability ratio k_f/k_b exceeds 10. In nature this ratio usually exceeds 10; it ranged from about 100 to 5000 for the 16 sites studied.

117. The determination of $\$$ (and hence Q_s , h_o , and h_x) is based on the base width of the levee L_2 , an evaluation of the distance from the landside levee toe to the effective source of seepage s , and the distance from the landside levee toe to the effective seepage exit x_3 . Determination of these latter three items (L_2 , s , and x_3) has been discussed earlier in this part. As previously stated the theoretical expressions for s and x_3 depend upon the extent, permeability and thickness of the top stratum and pervious foundation, and on whether a block or open drainage surface occurs at the extremities of the blanket. Formulas for computing s and x_3 are summarized in fig. 23.

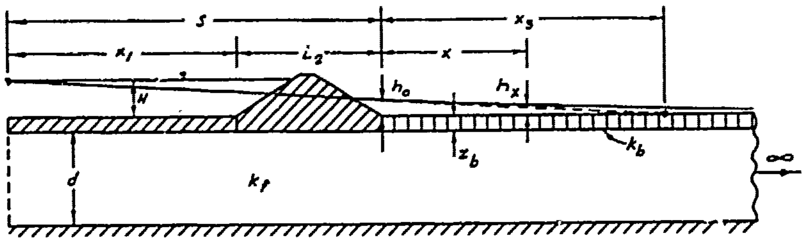
Basic Equations and Definitions

Seepage per unit length of levee........ $Q_s = f k_f H = k_f H \frac{d}{s + x_3}$

Head beneath top stratum at landside toe of levee .... $h_o = H \frac{x_3}{s + x_3}$

A factor..... .... $c = \sqrt{\frac{k_b}{k_f z_b d}}$

CASE 5

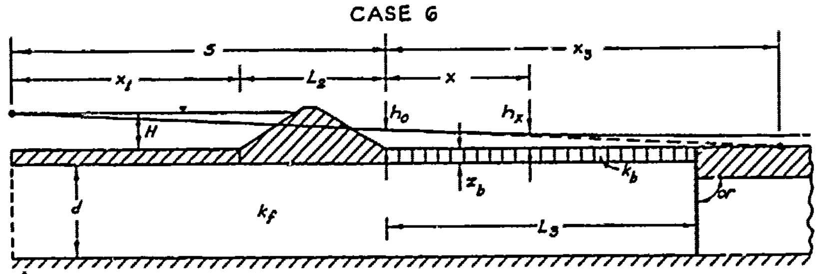


$x_3 = \frac{1}{c}$

$h_x = h_o \epsilon^{-cx}$

$(\epsilon = 2.718)$

CASE 6

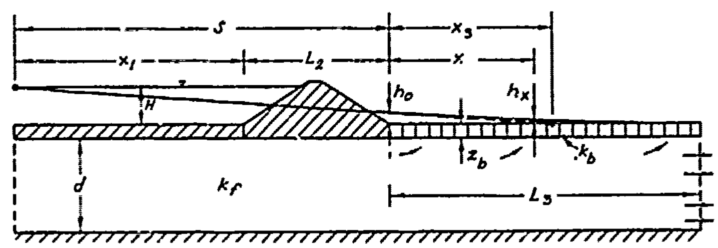


$x_3 = \frac{1}{c \tanh(cL_3)}$

$h_x = h_o \frac{\cosh c(L_3 - x)}{\cosh(cL_3)}$

$h_{x=L_3} = \frac{h_o}{\cosh(cL_3)}$

CASE 7



$x_3 = \frac{\tanh(cL_3)}{c}$

$h_x = h_o \frac{\sinh c(L_3 - x)}{\sinh(cL_3)}$

$h_{x=L_3} = 0$

Note:
x_1 can be computed from formulas for x_3 by inserting the length of riverside blanket L_1 for L_3 in the appropriate expression when riverside conditions are similar to the above landside conditions.

Fig. 23. Formulas for computation of seepage and substratum pressures for semipervious landside top stratum

118. Wherever both the riverside and landside top stratum are semipervious, the expression for the quantity of seepage per unit length of levee can be expressed by

$$Q_s = k_f H \mathcal{S} \tag{21}$$

where

$$\mathcal{S} = \frac{d}{s + x_3} \tag{29}$$

or

$$Q_s = k_f H \frac{d}{s + x_3} \quad . \tag{30}$$

The head h_o beneath the top stratum at the landside toe of the levee is expressed by

$$h_o = H \left[\frac{x_3}{s + x_3} \right] . \tag{31}$$

Equations 30 and 31 are valid for all conditions where the landside top stratum is semipervious.

119. The head h_x beneath the semipervious top stratum landward of a levee depends upon the head h_o and conditions landward of the levee. Expressions for h_x are given below for typical conditions encountered landward of levees (see fig. 23).

Case 5, $L_3 = \infty$

$$h_x = h_o \, \epsilon^{-cx} \tag{32}$$

$$(\epsilon = 2.718)$$

Case 6, L_3 = a finite distance, and a block exists at the end of the blanket

$$h_x = h_o \frac{\cosh c \, (L_3 - x)}{\cosh c \, L_3} \tag{33}$$

and

$$h_{x = L_3} = \frac{h_o}{\cosh c L_3} \tag{33a}$$

Case 7, L_3 = a finite distance, and an open drainage surface exists at the end of the blanket

$$h_x = h_o \frac{\sinh c (L_3 - x)}{\sinh c L_3} \tag{34}$$

$$h_{x = L_3} = 0 \tag{34a}$$

where

$$c = \sqrt{\frac{k_{bL}}{k_f \, z_{bL} \, d}} \; . \tag{7}$$

120. In order to simplify the determination of h_x for various values of x, a graph of the relationship between h_x/h_o and x/x_3 has been plotted in fig. 24 for $L_3 = \infty$, and for various values of x_3/L_3 for both a blocked and open exit at L_3. Once s, x_3, L_3, and the head h_o at the landside toe of the levee have been determined, the ratio h_x/h_o can be read from fig. 24 for any particular x/x_3; then h_x can be computed from h_x/h_o.

121. It can be shown from fig. 21 that for a given thickness and permeability of landside top stratum and foundation sand, the presence of a block landward of the levee increases the head at the toe of the levee and decreases the total seepage beneath the levee which would occur if there were no block and the top stratum were infinite in landward extent. However, the seepage at the toe of the levee would be heavier. Also the shorter the distance from the levee toe to the block L_3 the higher will be the head at the landward toe (see figs. 21 and 23). Where a block occurs landward of the levee and the corresponding x_3 has been determined from piezometric data, the x_3 that would have existed without the block can be determined from the factors given by the graph in fig. 25. This procedure was used for estimating the permeability of the landside top stratum at the piezometer sites where landside blocks exist.

122. Values of h_o and h_x given by the above equations are

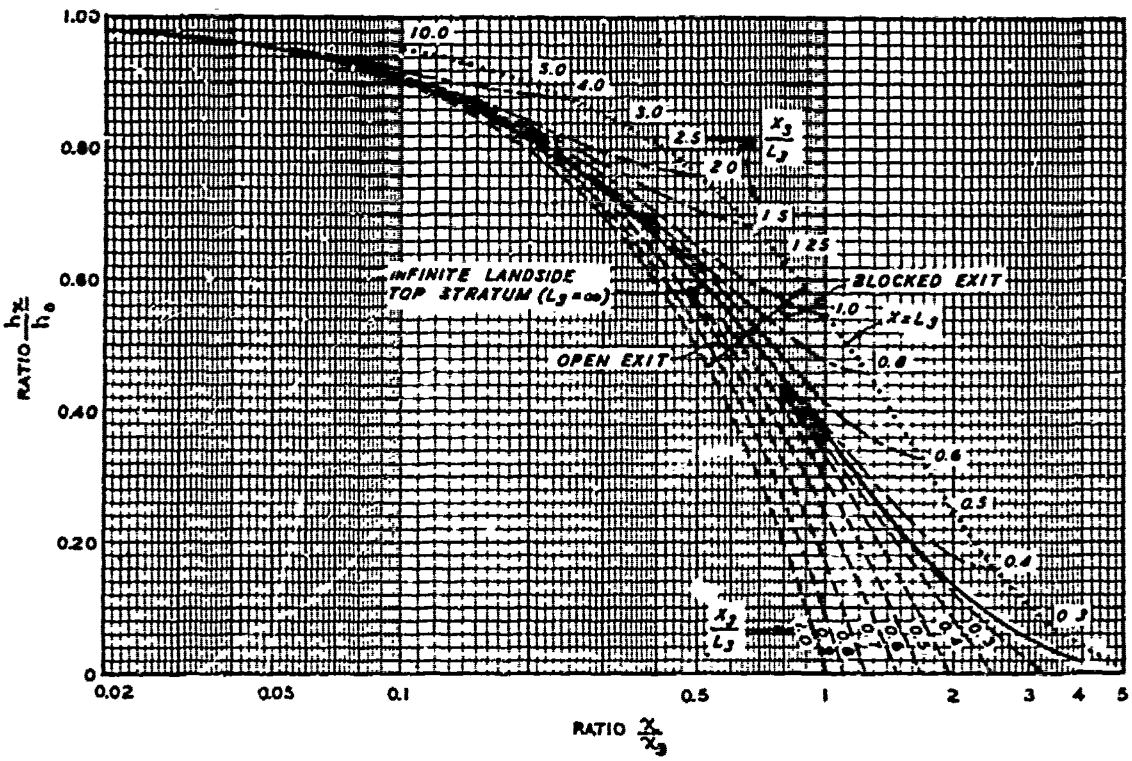


Fig. 24. Ratio between head landward of levee and head at landside toe of levee for levees founded on semipervious top stratum underlain by a pervious substratum

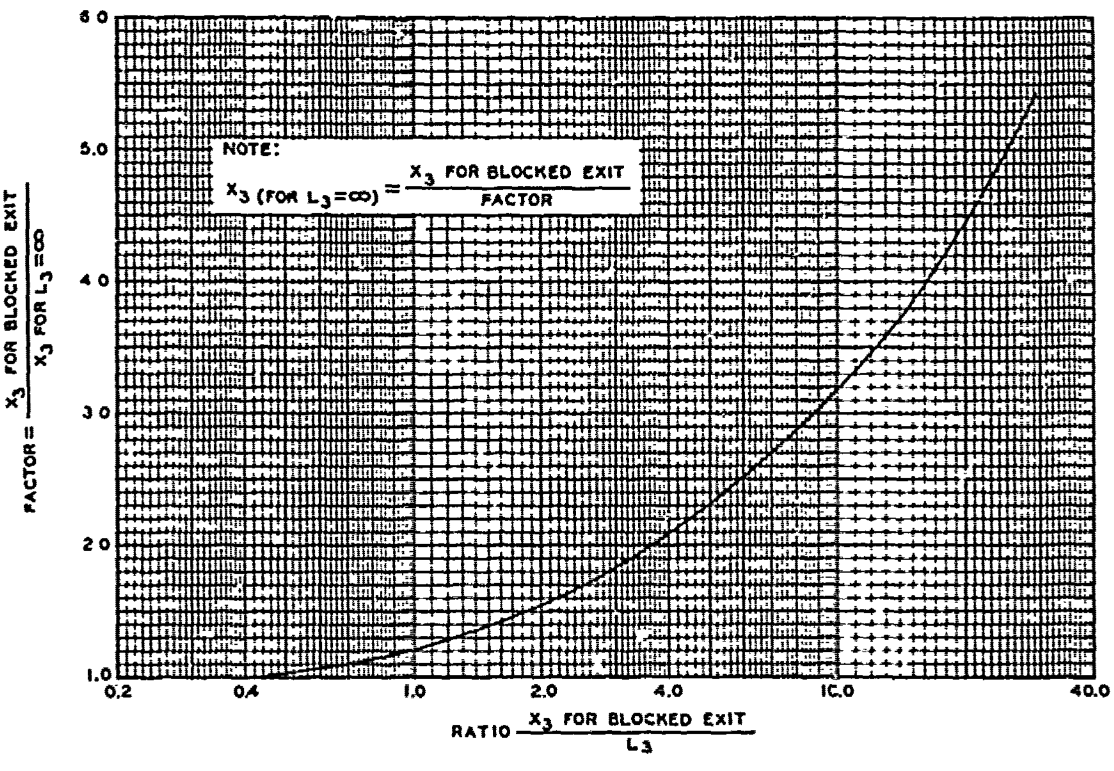


Fig. 25. Factor for computing x_3 for $L_3 = \infty$ from x_3 for blocked exits

hydrostatic heads at the middle of the pervious substratum; where k_f/k_{bL} is less than 100 to 500, values of h_o and h_x immediately beneath the top stratum will be slightly less than those computed from x_3 because of the head loss resulting from upward seepage through the sand stratum.

Estimation of substratum hydrostatic pressures from piezometer readings

123. The hydrostatic pressure at the landside toe of a levee at a project flood stage can be estimated from readings of a single piezometer at the landside toe by plotting readings observed during high-water periods vs corresponding overbank river stages. Extrapolation of such a plot will indicate whether the hydrostatic head at high river stages can be expected to equal or exceed h_c. If a value of h_c is likely to occur at river stages less than the design flood stage, control measures are indicated. If the piezometer readings have reached a maximum and remain constant for rising river stages, the critical hydrostatic pressure has been reached, and control measures are indicated if the project river stage is very much higher than the stage at which h_c occurred.

PART IV: INVESTIGATION OF UNDERSEEPAGE AT PIEZOMETER SITES

124. Basic to the over-all investigation of underseepage were the detailed study of geological and soil conditions and the measurement of substratum pressures and seepage flow by means of piezometers at 15 sites on the Mississippi River and one site on the Red River. The sites were selected where subsurface explorations had already been made and representative types of geological and top stratum conditions were known to exist. The sites included locations where no underseepage had occurred and where underseepage and sand boils had been a serious problem during the 1937 high water. The first six sections of this part describe the studies made, procedures and equipment used in the field, data-analysis methods, bases for seepage-control recommendations, and the manner of presenting the results of the studies. The remainder of Part IV summarizes the results of the studies by sites.

Types of Studies Made for Each Site

125. The studies made for each of the piezometer sites included:

a. Mapping. In general, three maps -- plan, topographic, and an aerial mosaic -- were prepared for each site. The plan maps, originally drawn to a scale of 1:10,000,* show the levee, river, and other pertinent geographic features; the general geology of the area; and locations of the piezometers that were installed. The topographic maps, prepared with 2-ft contour intervals and originally drawn to a scale of 1:2,400,** show the locations of the borings and piezometers in detail, the terrain, detailed geology, thickness and type of top stratum, and location of seepage and sand boils. The aerial mosaics also show the locations of the piezometers.

b. Field explorations. Both shallow and deep borings were made at each site to determine the soil stratification along the levee toe and along cross sections perpendicular to the levee. Most of the borings were of the auger or bailer type, with undisturbed Shelby tube samples of the

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* The plan maps in this report generally have a scale of 1:6666.
** The topographic maps in this report have a scale of 1 in. = 300 ft.

top stratum and upper fine sands obtained above the water table at some locations. The purpose of the borings was to assist in delineation of the geology of the site, provide data as to the thickness and type of top stratum and depth of the underlying sand, and furnish samples for mechanical analyses and permeability tests of the sands. From the boring logs generalized soil sections were developed and the thickness of the top stratum landward of the levee was determined. At Commerce and Trotters 54 pumping tests were performed in addition to the soil borings for the purpose of accurately determining the permeability of the underlying sand strata.

c. Geological studies. The general geology of each site as regards former river courses was taken from previous geological studies.[16] The detailed geology was established from a study of aerial photographs, topographic maps, and boring data.

d. Laboratory tests. Mechanical analyses and permeability tests were made on remolded sand samples to determine the gradation and permeability of the foundation sands. Values of D_{10} and k are plotted adjacent to the boring logs on the soil profiles.

e. Hydrostatic pressure measurements. Piezometers were located along the landside toe of the levee to determine the pressure beneath the top stratum, and along ranges perpendicular to the levee to measure the hydrostatic pressure beneath and landward of the levee and the distances to the effective seepage source and exit. Generally the tips of the piezometers were located immediately below the top stratum in the upper part of the underlying foundation sands. At some sites the tips were put down to a considerable depth in the sand stratum for the purpose of measuring head loss in the foundation sands in a vertical direction; at other sites some of the piezometer tips were located within the top stratum for measuring the loss in head within the top stratum.

f. Seepage measurements. The natural seepage emerging landward of the levee was measured at Gammon, Commerce, Trotters 51 and 54, Stoval, and Baton Rouge sites during the 1950 high water. At one site, Trotters 54, it was possible to compare the observed natural seepage during a high water before relief wells were installed to that observed during another high water (1951) when a system of relief wells was in operation. At the other sites the natural seepage measurements were compared with those computed from the permeability of the foundation and the hydraulic gradient beneath the levee.

Generally the natural seepage at the sites was measured at about the crest of the 1950 high water. It was impossible to collect all the seepage passing beneath the levee; but, in general, the area in which the seepage being measured was emerging could be delineated by field observations, and the approximate percentage of the total seepage passing beneath the levee and emerging in the delineated area could be estimated from hydraulic gradients. The rate of seepage was determined by measuring the flow in small ditches or culverts by means of a midget Gurley flow meter at locations where delineation of the seepage area was possible.

Piezometer Installations

126. The piezometers consist, in general, of commercial brass well screens 1-1/2 to 2 in. in diameter and 24 to 30 in. long. Riser pipes are of galvanized iron pipe 1 to 2 in. ID. They usually extend 2 to 3 ft above ground, and are protected either with a cylinder of concrete 6 in. in diameter poured around the upper part of the riser pipe or by means of guard posts. The piezometers riverward of the levee are protected by special devices such as shown on plate 140. The locations of all piezometers are given in table 1 of volume 2. The elevation of the project flood and the crests of the 1937, 1945, and 1950 high waters, the average elevation of the ground surface along the toe of the levee, and the maximum net head on the levee during the three high waters are recorded for each of the 16 sites in table 3.

Factors Evaluated and Methods Used in Analysis of Piezometric and Seepage Data

127. Factors L_1 , L_2 , and L_3 were obtained at each site from field surveys and existing maps. However, at some sites the value of L_3 was determined from a consideration of both geological information and piezometric gradient lines (see Gammon, paragraph 189). The depth and permeability of the pervious foundation, and thickness and permeability of the top stratum at the sites were estimated from field

Table 3

Summary of Maximum River Stages and Heads on Levee at Piezometer Sites

Site	Levee Station	Soil Section	Maximum High-water Stages: Project Flood	1937	1945	1950	Avg Ground Elevation at LS Toe of Levee	Maximum Net Head on Levee, ft: Project Flood	1937	1945	1950
Caruthersville, Mo.	26/0+00	A-A	287.2	283.5	277.9	277.4	269.0*	18.2	14.5	8.9	8.4
Gammon, Ark.	138/26+00	D-D	244.2	238.6	229.3	230.9	219.0*	25.2	19.6	10.3	11.9
Commerce, Miss.	23/10+75	H-H	220.2	218.9	207.4	206.7	198.0*	22.2	20.9	9.4	8.7
Trotters 51, Miss.	50/36+50	E-E	208.7	204.2	193.4	194.0	183.0*	25.7	21.2	10.4	11.0
Trotters 54, Miss.	54/1+05	M-M	207.7	203.7	192.9	193.8	180.0*	27.7	23.7	12.9	13.8
Stovall, Miss.	77/38+00	A-A	194.3	191.3	178.9	179.4	164.5*	29.8	26.8	14.4	14.9
Farrell, Miss.	81/24+00	A-A	192.1	190.8	178.5	174.8	166.0*	26.1	24.8	12.5	8.0
Upper Francis, Miss.	39+00	B-B	185.0	178.4	170.6	168.3	160.0**	25.0	18.4	10.6	8.3
Lower Francis, Miss.	130+00	C-C	182.6	177.5	170.1	167.6	154.0**	28.6	23.5	16.1	13.6
Bolivar, Miss.	2199+75	D-D	167.2	159.3	151.8	147.5	139.0**	28.2	20.3	12.8	8.5
Eutaw, Miss.	2860+00	D-D	163.1	152.2	146.4	141.2	132.0**	31.1	15.2†	9.4†	9.2
L'Argent, La.	3542+33	B-B	90.0	81.1	77.8	75.4	60.0**	30.0	21.1	17.8	15.4
Hole-in-the-Wall, La.	3618+95	D-D	89.5	80.7	77.4	75.4	66.0**	23.5	14.7	11.4	9.4
Kelson, La.	2700+71	B-B	53.7	49.9	50.8	48.7	32.0**	21.7	17.9	18.8	16.7
Baton Rouge, La.	97+42	C-C	48.0	44.1	44.6	42.4	25.5*	22.5	18.6	19.1	17.7
Cotton Bayou, La.	826+49	B-B	95.4 99.2††	-----	-----	86.5	82.0**	13.4 17.0	----	----	4.5

* Feet mgl.
** Feet msl.
† Based on ground elevation along landside toe of levee before berm was built.
†† Interim flood.

explorations, pumping tests, laboratory tests, and analysis of piezometric and seepage observations. The distance to the effective seepage entrance and exit, substratum pressures landward of the levee, and hydraulic gradients beneath and landward of the levee were determined from piezometric data. Seepage flow beneath the levee was estimated from hydraulic gradients and characteristics of the pervious substratum. Predicted hydraulic grade lines at project flood stages were based on theoretical formulas discussed in Part III, extrapolation of observed data, and on borrow pit and levee conditions, and seepage control measures as they existed in 1950. Predicted values of h_o and upward gradients were based on extrapolations of readings of piezometers beneath the top stratum at the landside toe of the levee or existing seepage berm

128. River stages during significant high-water periods were obtained from gages installed at the sites. Gages and piezometers were read at two- to three-day intervals during significant high-water periods. The river stage and piezometers at any one site, though not read simultaneously, were always read the same day; however, the rate of rise or fall of the river was sometimes of such magnitude that the lack of simultaneous reading of river stage and piezometers probably caused some error in the relation between the readings. The relationship between overbank river stages at each site and those at a nearby permanent gage on the Mississippi River are shown in volume 2. The graph on plate 14 is an illustration of this relationship at the Caruthersville, Mo., site. The relationship is based on gage readings obtained at the site and at the nearest permanent river gages during various rising river stages. Because of differences in the slope of the water surface of the river for different floods, the correlation shown is only approximate. The relation between permanent gages and those at the sites was extrapolated to the project flood, and therefore may not be as exact for river stages in excess of those already experienced.

129. Where the ground landward of the levee is essentially level, tailwater was assumed equal to about the average elevation of the ground. Water was ponded over a part of the area landward of the levee at some of the sites and in those cases the elevation of water surface was recorded;

e.g., Commerce, plate 48. At other sites the exact elevation of the tailwater was not determined and had to be estimated.

Borrow pit conditions

130. The width of borrow pits and type and thickness of material in the bottom of the pits are summarized for each line of piezometers considered in the discussion of each site later in this portion of the report.

Seepage source and exit

131. At each site at least one line of piezometers was installed perpendicular to the levee. The hydraulic grade line in the substratum sands and the distances from the landside toe of the levee to the effective seepage source and exit were determined from readings of these piezometers. The hydraulic grade line at the crest of the high water is plotted for each piezometer line perpendicular to the levee at each site, e.g., plates 12 and 13. The distances to the effective seepage source and exit were computed from equations 10 and 14, respectively, using data from piezometers located beneath the levee (and berm if present) and with tips in the substratum sands. When there were three or four piezometers beneath the levee, s and x_3 were determined graphically. Unless otherwise noted, s and x_3 are referred to the landside toe of the levee or berm.

132. Computation of s and x_3 (see paragraphs 93 and 97) requires the use of the average head in the pervious substratum at the points of measurement. Only at a few sites such as Commerce, Trotters 51, and Trotters 54 were piezometers installed sufficiently deep in the pervious foundation to obtain the average pressure in the sands. Data at these sites indicate that: (a) the head immediately beneath the top stratum under the middle portion of the levee is equal to that at any depth in the pervious substratum, and (b) where there is a significant flow of upward seepage the head immediately beneath the top stratum at the landside toe of the levee is somewhat less than the average head in the sand stratum at the levee toe. The difference in head developed at the landside toe of the levee immediately beneath the top stratum and at or near the mid-depth of the substratum sand at the above sites is plotted

for various net heads in fig. 26. At these sites the approximate average difference in excess head at the landside levee toe as measured immediately beneath the top stratum and near the center of the pervious substratum is about 14 per cent of the net head on the levee. At sites where no deep piezometers were installed and where the line consisted of two shallow piezometers, one beneath the levee and one at the levee toe (or berm toe where present), it was necessary to estimate the average head in the substratum sand at the levee toe from the reading of the shallow piezometer at the toe prior to computing s and x_3. At sites similar to the above three sites the average head at the levee toe was estimated as the head in the shallow piezometer at the toe plus the average difference in head between deep and shallow piezometers for the same net head as recorded at Commerce and the two Trotters sites; see fig. 26. At sites where the seepage flow landward of the levee appeared to be less than that at the Commerce and Trotters sites, the average head at the levee toe was assumed to be the head at the shallow piezometer plus 1/8 to 1/2 of the average difference in head shown in fig. 26.

133. Distances to the effective seepage source and exit determined on selected days during the high-water period are plotted vs the river stage occurring on that day; e.g., fig. 27 for Caruthersville. From such plots, values of s and x_3 were extrapolated to the project flood. The extrapolation of s to the project flood was based on observed trends during previous high waters with consideration given to the possibility of changes in the riverside borrow pits. The extrapolation of x_3 to the project flood was obtained by solving equation 31 for x_3, using the values of s, H, and h_o estimated to exist at the project flood. In so doing, h_o was taken as the average head in the substratum sand at the landside toe of the levee. A curve then was drawn through the observed seepage exits (plotted against the corresponding river stages) to the extrapolated x_3 at the project flood. The curves relating x_3 and river stage are of the following types:

<u>a</u>. <u>x_3 independent of river stage</u>. A constant x_3, vertical line on the plot of river stage vs x_3, e.g., fig. 27, indicates that the resistance to the flow of seepage

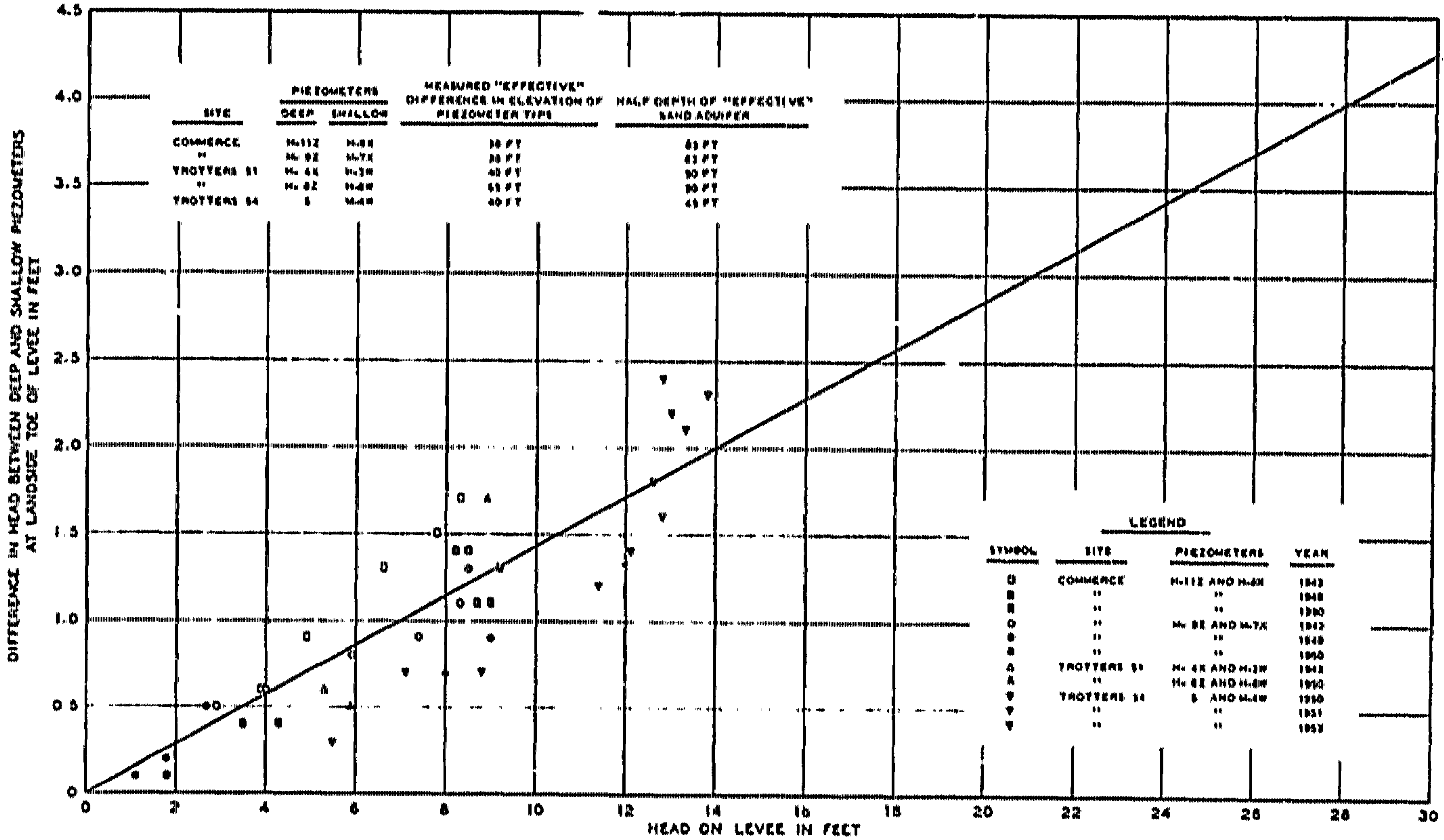

Fig. 26. Difference in head at deep and shallow piezometers at landside toe of levee

either landward or up through the natural blanket remains constant for various river stages.

b. x_3 decreases with rising river stage. A decrease in x_3 with rising river stage indicates a decrease in the resistance to seepage up through the landside top stratum. This was common at a number of sites at relatively low river stages. For example, the occurrence of sand boils provides additional outlets for seepage, thereby decreasing the resistance offered by the natural blanket to seepage through the top stratum and thus decreasing the distance to the effective seepage exit. On the x_3 vs river stage plots, the point at which x_3 first starts to decrease, as the river stage increases, corresponds approximately to the stage at which sand boils were first observed (e.g., line M , Trotters 54, fig. 32).

c. x_3 increases with rising river stage. An increase in x_3 with rising river stage indicates an increase in the resistance to the flow of seepage landward. This phenomenon may be explained as follows: At the beginning of overbank river stages the natural water table may be low and there may be a large volume of unfilled porous space in the ground into which seepage may readily flow and which in a sense acts as a drainage reservoir for underseepage. For this condition the apparent seepage exit, as previously defined, may be quite close. As the subsurface storage becomes filled in the vicinity of the landside toe, the phreatic line comes in contact with the bottom of the top stratum and seepage has to either flow into storage areas farther landward or force its way up through the top stratum. In either case the resistance to the flow of seepage landward is increased, which will increase the distance to the effective seepage (drainage) exit; e.g., line E, Trotters 51, fig. 31. If continued rising river stages cause the development of sand boils, such boils will provide a form of drainage that will result in a subsequent decrease in x_3 . It should be noted that prior to such time as the foundation beneath the levee becomes fully saturated, a condition of truly artesian flow will not exist and, as a result, values of s and x_3 computed from equations 10 and 14 may not be reliable. At certain lines and sites, values of s and x_3 could not be determined because the piezometers were sluggish and/or gave erratic readings.

134. There are numerous single piezometers along the toe of the levee at the various sites. Although the data from these piezometers were used in estimating the severity of underseepage, they were not

used to estimate values of s and x_3 .

Thickness and permeability
of pervious substratum

135. The total thickness of the pervious substratum was determined from deep borings. The effective thickness d was determined as that thickness of the principal seepage-carrying sand stratum below the top stratum. (See definitions of thickness and permeability in Part V, paragraph 651.) The thicknesses of very fine sand strata of low permeability were transformed into reduced equivalent thicknesses with the same permeability as that of the principal seepage-carrying stratum. The permeability of the effective pervious substratum was determined using various methods; the values so obtained at each site are tabulated in the discussion of each site. Methods of determining k_f are explained in Part III of this report. Those used in the analysis of data from the piezometer sites are summarized as follows:

a. Laboratory permeability tests. Laboratory permeability tests were made on samples from deep borings at or near the landside toe of the levee, from which data k_f was estimated by equation 9. Where the borings did not fully penetrate the substratum, k_f for the part of the substratum beneath the boring was estimated from values of k_f obtained from the deeper samples in the borings. This appears reasonable because it has been found that the permeability of the substratum generally tends to increase with depth in the Lower Mississippi River Valley. The results of the laboratory permeability tests on remolded samples are shown adjacent to the logs of borings in volume 2.

b. Grain-size data. The permeability of the foundation was also estimated from the D_{10} size of the sand in each stratum comprising the substratum by a procedure similar to a, above, except that k_f was determined by means of the empirical relationship between D_{10} and k_H shown in fig. 17. The relationship between the permeability of remolded samples of sand, taken with a bailer, as determined in the laboratory and D_{10} is shown for the various sites on plate 244. After this plate had been prepared, numerous field pumping tests were conducted from which the relationship between field permeability k_H and D_{10} was obtained as shown in fig. 17. It is believed that a better estimate of k_f can be obtained from fig. 17 than from plate 244, and therefore fig. 17

was used in estimating k_f from grain-size data. Values of D_{10} as obtained from mechanical analyses are plotted adjacent to the boring logs in volume 2.

c. Seepage and piezometric data. These data were used in estimating k_f by means of equation 2a at the six sites where the seepage emerging landward of the levee was measured.

d. Field pumping tests. Pumping tests were conducted at Commerce and Trotters 54 (see Appendix C) to determine the permeability of the sand. The results of these tests are given in the table of analysis of piezometric and seepage data for these sites.

e. Well flow data. At Commerce and Trotters 51, systems of relief wells were in operation during the 1943 high-water period, and values of k_f were computed from the well flow data. Similar computations were made for the relief well systems at Trotters 54 from data obtained during the 1943 and 1951 high-water periods. The permeability of the sand substratum was computed from equation 2b by replacing Q_A with the well flow per unit length of levee and M_A with the slope of the hydraulic grade line landward of the well system.

f. Best estimate. A best estimate of k_f is given for each site. This estimate is based on the k_f values obtained by methods a-e, inclusive, with emphasis on results from methods b, c, d, and e.

Thickness of top stratum

136. The top stratum was transformed into a blanket of uniform vertical permeability of a specific thickness as illustrated in fig. 13. The thickness transformation factors used were approximately the same as given in table 1.

137. Blanket formulas show that for a uniform top stratum infinite in landward extent 64% of the seepage flow rises to the surface between the landside levee toe and the effective seepage exit. Therefore, borings made between the toe of the levee and the effective seepage exit were given more weight than those farther landward when estimating z_{bL}. All borings in an area with a length, parallel to the levee, equal to the exit length were considered in determining the average thickness of the top stratum at a given piezometer line. At sites where thin spots in the top stratum are present, because of either natural deposition or man-made ditches, it was necessary to assign values of z_t as well as z_{bL},

as such thin areas determine the amount of excess head that may safely be allowed to develop. The same thickness transformation factors were used for z_t and z_{bL}.

Permeability of top stratum

138. The permeability of the top stratum k_{bL} was estimated by means of blanket formulas and measurements of natural seepage as described below.

a. Blanket formulas. Where the top stratum was uniform and infinite in landward extent, k_{bL} was estimated by means of equation 5 using the best estimate of z_{bL} and k_f for each piezometer line. However, at some sites the top stratum does not extend infinitely landward but instead the presence of a clay-filled channel effectively prevents seepage farther landward and results in a blocked exit; e.g., case 6 of fig. 23, and plates 20 and 27. The value of k_{bL} for the portion of the blanket between the levee toe and block L_3 can be estimated from equation 6 or more easily from equation 5 after the distance to the effective seepage exit (x_3 for finite L_3) as indicated by piezometers is converted to a corresponding x_3 for $L_3 = \infty$ using the factors shown in fig. 25. Values of k_{bL} computed from blanket formulas are given for each piezometer line in the tabular summary for each of the piezometer sites.

b. Seepage measurements. At the six sites where the quantity of seepage emerging in a given area landward of the levee was measured, k_{bL} was also computed from equation 4.

c. Best estimate. At 11 sites k_{bL} could only be estimated from blanket formulas which consequently gave the best estimate. At the six sites where natural seepage was measured the best estimate of k_{bL} was obtained by proportioning results obtained from a and b above, giving more weight to results from method b. Values of k_{bL} are given for both the 1950 high-water period and the project flood; the latter values of k_{bL} are estimates of conditions without any additional seepage control measures. It should be noted that at many sites k_{bL} will increase as the river rises and sand boils develop, and therefore the predicted values of k_{bL} at the crest of the project flood usually exceed those observed in 1950.

Permeability ratio k_f/k_{bL}

139. The ratio of the permeability of sand substratum to that of the top stratum is given in the tabular summary for each site. The first value listed in the table is the best estimate of k_f/k_{bL} obtained by the following methods:

a. Blanket formulas. Blanket formulas were used to estimate the permeability ratio. Where the landside top stratum is infinite in landward extent, equation 5 can be written as follows:

$$\frac{k_f}{k_{bL}} = \frac{x_3^{\,2}}{z_{bL}\,d} \tag{5a}$$

where z_{bL} and d are determined from boring data and x_3 is obtained from piezometric data as previously described. Where a block occurred landward of the levee, the observed x_3 was converted to an equivalent x_3 for $L_3 = \infty$ (from fig. 25) as previously described.

b. Natural seepage measurements. The permeability ratio was also obtained by dividing the value of k_f by the value of k_{bL} obtained from natural seepage measurements and equation 4.

Seepage beneath levee Q_s

140. The seepage beneath the levee per 100 ft of levee was computed for the crest of the 1950 flood and was also computed for the project flood from equation 30 using the best estimated values of d , k_f , s , and x_3 for the project flood. Values of seepage Q_s/H in gpm per ft net head per 100 ft of levee are shown in the tables. The severity of seepage at the sites during the 1950 high water has been based on the following arbitrary classification.

Q_s/H gpm per 100 ft of Levee	Severity of Seepage
>10	Heavy
5-10	Medium
<5	Light

Piezometer readings versus river stage

141. Readings of selected piezometers at the landside toe of levee or berm were plotted against the corresponding river stages for different high waters; e.g., Caruthersville, plates 14 and 15. The ratio of the head h_o at the different piezometers to the net head H on the levee was computed and is shown on the curves drawn through the plotted data. Also shown are the estimated hydrostatic heads for the project flood; these heads were based on extrapolations of observed data and consideration

of the computed maximum possible substratum heads. Values of h_c were computed by multiplying the transformed thickness of top stratum z_t by 0.85 (assumed i_c). A value of 0.85 was assumed for i_c to give a degree of uniformity to the resulting estimated maximum substratum pressures. As discussed later in the report, it was found that i_c appears to vary widely at several of the sites studied where sand boils developed; generally, i_c was equal to or less than 0.85. The maximum piezometer readings were computed by adding h_c to either the ground elevation at the piezometer or the tailwater elevation where the ground at the piezometer was submerged. Ditches frequently are critical as regards underseepage; therefore, some piezometers were installed on both sides of (but not in) existing ditches. The head presumed beneath the bottom of the ditch (Caruthersville, plate 15) was taken as the average of the nearby piezometers.

142. A table entitled "Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee" is included for each site. This table contains a summary of z_t and h_c, and values of h_o for the crest of the 1950 flood and the estimated h_o at the project flood. Gradients through the top stratum and related severity of seepage also are summarized in this table.

143. The hydrostatic head h_o at various piezometers landward of the levee was estimated for the project flood as described below. The letter shown in the table by each h_o estimated at the project flood corresponds to one of the following methods of estimation.

a. Where piezometer readings continued to increase linearly with river stage and h_o extrapolated to the project flood was less than h_c (= 0.85 z_t), the h_o that will develop at the project flood was taken to be the value obtained by the above extrapolation (see piezometer 1, line D, Caruthersville, plate 14).

b. Where piezometer readings reached a maximum and then remained constant during a period when the river continued to rise it was assumed that the substratum pressure would not rise above such value regardless of head on the levee and that h_o at the project flood stage would be the same as the above observed maximum (see piezometer C-5, line C, Gammon, plate 30).

c. Where piezometer readings increased with rising river stages but it appeared h_o would become equal to h_c before the project flood stage was reached, the curve fitting the data was extrapolated to $h_o = h_c$ (for $i = 0.85$). See piezometer 11, line D, Caruthersville, plate 15.

d. Where piezometer readings were available for several high waters and it appeared that the maximum head recorded by the piezometer at the project flood would be less than h_c , the curve fitting the data was extrapolated by eye to the project flood.

144. Landside piezometer readings remaining constant over a period when the river stage continued to rise indicated that the critical head beneath the top stratum had developed and that sand boils, with severity dependent on the height of river stage above that causing h_c , had developed at the site before the project flood crest was reached.

145. The initial portion of plots of piezometer readings vs river stage indicate that artesian heads usually did not develop beneath the top stratum until after water had been against the levee for several days. This initial lag is attributed to the large volume of storage above the initial ground-water table landward of the levee which must be filled before the hydrostatic head in the pervious substratum can rise above the ground surface.

Hydraulic gradient beneath levee

146. Hydraulic gradients beneath and landward of the levee were plotted for each site at a river stage equal to about 1/3 or 1/2 the maximum stage, at maximum stage, and when the river had fallen to about the natural ground surface. The gradient in the middle of the pervious aquifer was estimated for a project flood stage for one of the piezometer lines at each site. The gradient lines estimated for a project flood were based on predicted values of s and x_3 at a project flood stage and values of k_f/k_b , z_{bL} , and d .

Basis for Evaluation of Seepage Problem, and Recommendations Regarding Seepage Control Measures

147. The adequacy of existing berms (and the relief well system at Trotters 54) was evaluated for each site. The evaluation was based on

the criteria given in paragraph 675.

148. Where the analysis of piezometric and seepage data from the sites indicates that substratum pressures landward of the levee have exceeded or would exceed maximum allowable heads, seepage control measures are recommended unless an existing seepage berm or sublevee is considered to be adequate to safeguard the levee. Design values for each site and/or subreach therein are given in tables titled "Summary of Analysis of Piezometric and Seepage Data, and Average Design Values." The distances to the source of seepage selected for design values are those estimated at the project flood; the effective seepage exits are those determined from piezometer readings, modified where necessary to conform with the values of z_{bL} , k_{bL} , d , k_f , and L_3 selected for the indicated reaches of levee. The selection of design values for x_3 , k_{bL} , and k_f/k_{bL} was predicated on the intent that the resulting seepage control measures would prevent the landside top stratum from becoming more pervious than it was observed to become during the 1950 high water as a result of seepage and sand boils. If the design of control measures had been based on values (x_3 and k_f/k_{bL}) predicted for the project flood without additional control measures, the design of the new control measures would not be compatible with top stratum conditions until after the seepage situation became worse than that which existed during the 1950 high water.

Methods of Presenting Results of the Studies

149. The results of the studies are presented in the remainder of this part and on the plates in volume 2. The discussions of each site include the following information:

- a. Description of the site which embraces (1) maps and an aerial mosaic, (2) a history of the underseepage, (3) underseepage control measures installed, and (4) features of the piezometer installation.
- b. Geology of the site and soil conditions which describe (1) relation of underseepage to geology, (2) soil profiles and piezometer lines, and (3) characteristics of top stratum and foundation.

c. Analyses of piezometric and seepage data by (1) estimation of numerical values of factors L_1 , L_2 , L_3 , x_1 , s , x_3 , d , k_f , Q_s , h_o , and i_c involved in seepage, (2) study of substratum hydrostatic pressures and hydraulic gradients beneath and landward of levee, and (3) comparison of measured and computed seepage flows.

d. Evaluation of seepage problem and recommendations regarding control measures if any are indicated.

150. The plates of volume 2 comprise maps (paragraph 149a), geological and soil profiles, and plots of piezometer data. The plan and topographic maps present the history and location of underseepage during the 1937 and 1950 high waters; the topographic maps also show existing seepage control measures. On the plan maps the surface geology at each site is delineated by means of various hatchings and colors. The type and thickness of the top stratum are shown in a similar manner on the topographic maps. (The legends for the symbols, hatchings, and colors are given on plate 1.) The location of underseepage and sand boils with respect to geological features and top stratum characteristics is also shown on the topographic maps. The soil profile plates present geological and soil profiles in color, all available logs of borings,* laboratory data, and potential sources of underseepage. Locations where seepage observation points were established can be found on the topographic maps together with the date, river stage, and amount of seepage observed at these points.

Caruthersville, Missouri

151. Caruthersville was selected as a site for study and the installation of piezometers primarily because of its record of heavy underseepage and sand boils along certain reaches of the levee, whereas underseepage had not been a problem at other locations in the area. The top stratum landward of and along the levee section investigated may be

* The classifications of soil types shown on the boring logs were based on the LMVD soil classification triangle included on plate 1, since all borings were made before development of the Unified Soil Classification System.

divided into four basic types and thicknesses: (a) 10 ft of clay overlain by 5 ft of silt; (b) 2 to 5 ft of clay overlain by 5 ft of silt; (c) 3 to 6 ft of silt; (d) 15 ft of silt.

Description of site

152. The site is located along the west bank levee of the Mississippi River near Caruthersville, Mo., and extends from about sta 25/0 to 26/20. Plans of the site, river, borrow pits, piezometers, borings, and the locations of underseepage and sand boil areas are shown on plates 2 and 3; plate 4 is an aerial mosaic of the site. The Mississippi River is approximately 1500 ft from the levee. Riverside borrow pits approximately 10 ft deep and 500 ft wide extend the full length of the section of levee under study. The levee in the vicinity of sta 26/0 has a net height of about 18 to 20 ft. River stages at this site can be estimated from the Mississippi River gage at Cairo and the graph on plate 14.

153. History of underseepage. During the 1937 high water, H of 14 to 15 ft caused heavy underseepage and numerous pin boils along the levee toe and for a distance 1000 ft landward between sta 25/0 and 26/30; sand boils were observed between sta 25/51 and 26/8. In 1939 a relatively thin seepage berm approximately 100 ft wide was constructed along the landside toe of the levee, as shown on plate 3.

154. During the 1950 high water, maximum H was approximately 8 to 9 ft; sheet seepage occurred along the berm toe and for a distance of 2000 ft landward between sta 25/0 and 26/5; and medium underseepage and sand boils were observed between sta 26/5 and 26/20 (plate 3).

155. Piezometer installation. In 1943 lines of piezometers were installed perpendicular to the levee at sta 26/0 (line A), and at sta 26/10 (line C); piezometers were also placed along the toe of the seepage berm from sta 25/0 to 31/0 (line D). The tips of all piezometers were installed in clean sand immediately beneath the top stratum. Piezometer readings were obtained during high waters in 1943 and 1950.

Geology of site
and soil conditions

156. The surface geology is shown on plates 2 and 3. Plate 2 shows the locations of former river courses in the area and natural levee

deposits which blanket the area. A more detailed presentation of the character and thickness of the top stratum in the vicinity of sta 26/0 is depicted on plate 3, which also shows the location of underseepage with respect to geological features.

157. The site is situated mainly in an area of point bar deposits laid down as the river gradually enlarged a meander loop (plate 2). During former river course 11 a chute developed, approximately 1800 ft wide, which subsequently was filled with silts and clays ranging in thickness from 10 to 20 ft (plate. 7 and 8). The fine-grained top stratum of the point bar deposits in course 10 consists of a stratum of silty clay 2 to 6 ft thick overlain by 3 to 5 ft of natural levee deposits of sandy silt and silty sand. The top stratum downstream of course 11 consists of 4 to 10 ft of silty sand and sandy silt. The thickest top stratum deposits are found in the old channel of course 11.

158. Relation of underseepage to geology. At this site underseepage appears to be associated with the insufficient thickness and silty character of the top stratum. During the 1950 high water the heavier underseepage and sand boils were observed at and immediately downstream of the downstream edge of the course 11 chute (plate 3). This can probably be attributed to the thinness and silty nature of the top stratum in this area and to the fact that foundation sands are probably exposed to a greater extent in the riverside borrow pits opposite this location than along other reaches of the borrow pit in the area.

159. Soil profiles and piezometer lines. Soil profiles at piezometer lines A, C, and D are shown on plates 5, 6, and 7, respectively. Deep borings to Tertiary show the existence of a pervious sand stratum about 100 ft thick extending from the river beneath and landward of the levee. A generalized soil profile, with the same horizontal and vertical scales, extending from the river to landward of the levee along line B is shown on plate 5.

160. Piezometer line A was located in the center of a chute of course 11 and in an area where the heaviest underseepage had been observed during the 1933 and 1937 high waters (plate 3). Piezometer

line C was installed perpendicular to the levee at the edge of course 11 chute where the top stratum was quite thin (5 to 7 ft) and consisted largely of silty sand and sandy silt. Individual piezometers were located along the landside toe of the present seepage berm as indicated on plate 7.

161. Piezometer 1 at sta 25/0 was installed in former river course 11 (see plate 2) at a point where the top stratum consists of about 9 ft of clay overlain by 5 ft of natural levee deposits. Plates 3 and 7 show that the borrow pits opposite piezometer 1 are blanketed by 3 to 5 ft of clay. Piezometer 13, south of Caruthersville at approximate sta 31/0, was deliberately located at a point where no significant underseepage had ever been reported (plate 3). The boring for this piezometer revealed essentially a silt top stratum approximately 15 ft thick. A cross section of the borrow pit opposite piezometer 13 indicates that the bottom of the pit at this location is probably blanketed by at least 5 to 6 ft of silt.

Analysis of piezometric and seepage data

162. River stage and piezometer readings obtained during the 1943 and 1950 high waters are plotted on plates 9-11. These high waters created a maximum H of about 8 to 9 ft. Piezometric gradients in the pervious substratum beneath the levee along piezometer lines A and C at selected river stages during these high-water periods are shown on plates 12 and 13, which also show the hydrostatic head along the toe of the levee as measured by line D piezometers. From these plates it can be seen that excess heads of about 0.5 to 1.5 ft developed at the berm toe at lines A and C during these floods. These heads decreased rapidly farther landward, and little or no excess head existed 300 ft landward of the berm toe. The greatest head at the toe of the berm was estimated to be about 5 ft at sta 26/6 during the 1950 high water (line D, plate 13). Plates 12 and 13 also show that the hydraulic gradient beneath the levee and berm was essentially a straight line at lines A and C. A summary of information pertaining to the site and results of analyses of piezometric and seepage data are given in table 4.

Table 4

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Caruthersville, Mo., Site

Factor	Line A 1943 Flood	Line A 1950 Flood	Line A Project Flood	Line C 1943 Flood	Line C 1950 Flood	Line C Project Flood	Average Design Values Sta 25/40 to 26/9	Average Design Values Sta 26/9 to 26/16	Average Design Values Sta 26/16 to 26/22
River stage (crest)	276.2	277.4	287.2	276.2	277.4	287.2	287.2	287.2	287.2
Average el of ground or tailwater	268.0*	266.0*	268.0*	270.5	270.5	270.5	268.0	270.5	270.5
Head on levee (H)	8.2	9.4	19.2	5.7	6.9	16.7	19.2	16.7	16.7
Piezometers used in analysis	3 & 4	4 & 5**	-----	8 & 9	8 & 9	---	-----	-----	-----
Riverside borrow pit, width, ft	300	-----	-----	300	-----	-----	300	300	300
Top stratum	0-5 ft clay 2-5 ft Si Sd	-----	-----	10 ft Si Sd	-----	-----	-----	-----	-----
Average stratum	5 ft Si Sd	-----	-----	10 ft Si Sd	-----	-----	5 ft Si Sd	Sand	Sand
Distance from riverside levee toe to river (L_1)	1200	-----	-----	1200	-----	-----	1200	1200	1200
Base width of levee (L_2)	330	-----	-----	280	-----	-----	330	300	300
Landward extent of top stratum (L_3)	-	-----	-----	-	-----	-----	-	-	-
Distance to effective seepage source (s)	775	560	480	810	560	480	500	500	500
Effective length of riverside blanket (x_1)	445	230	150	530	280	200	170	200	200
Distance to effective seepage exit (x_3)	250	240	200	200	200	180	230	170	230
Effective thickness of sand substratum (d)	100	-----	-----	100	-----	-----	100	100	100
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1500	-----	-----	1500	-----	-----	1500	1500	1500
Laboratory permeability tests	1150	-----	-----	1150	-----	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	-----	-----	-----	-----	-----	-----	-----	-----	-----
Seepage and piezometric data	-----	-----	-----	-----	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	-----	-----	-----	-----	-----	-----	-----
Top stratum, type	Clay silt	-----	-----	Silty sand	-----	-----	Silty clay	Silty sand	Silty sand
Effective thickness for seepage analysis (z_{bL})	6.5	-----	-----	7.5	-----	-----	7.0	4.5	8.0
Critical thickness (z_t)							7.0	4.5	8.0
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	15	17	25	25	25	35	-----	-----	-----
Piezometric data and blanket formulas	16	17	24	28	28	35	-----	-----	-----
Piezometric data and seepage measurements									
Permeability ratio (k_f/k_{bL})	100	90	60	60	60	45	75	60	60
Blanket formula	96	89	62	53	53	43	-----	-----	-----
Natural seepage measurements	-----	---	-----	-----	-----	-----	-----	-----	-----
Natural seepage beneath levee									
Q_s, gpm/100 ft of levee	180	160	625	110	225	560	-----	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	22	18	33	19	33	33	-----	-----	-----

* Average of natural ground and bottom of ditches.
** Piezometer 3 was sluggish in 1950.

163. Source of seepage. Seepage can enter the pervious foundation at the Caruthersville site through the river bank approximately 1500 ft from the levee, and through riverside borrow pits from which the top stratum has largely been removed in the vicinity of the central part of the piezometer system. The thickness and character of the remaining top stratum materials in the borrow pits in the vicinity of sta 26/0 are illustrated by the colored hatching on plate 3.

164. Values of s as determined at piezometer lines A and C for various river stages during the 1943 and 1950 high-water periods are plotted in fig. 27. These values indicate that seepage enters the sand substratum primarily through the borrow pits, even though several feet of silt remain in the pits (plates 5 and 6). (The source of seepage as determined graphically at lines A and C is shown on plates 12 and 13.) The distance to the effective source of seepage at lines A and C was

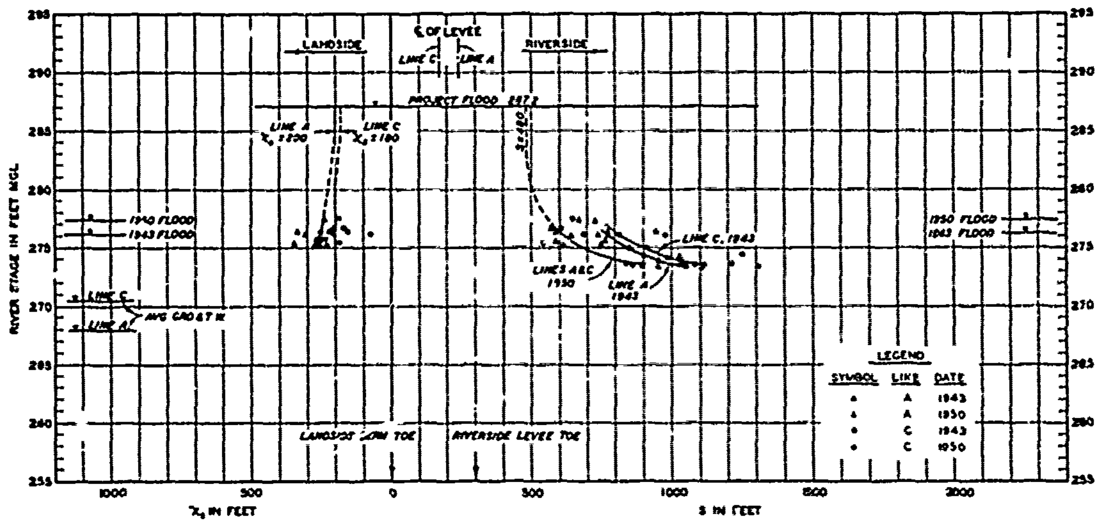

Fig. 27. Distances to effective seepage source and exit. Caruthersville, lines A and C

about 800 ft at the crest of the 1943 flood and about 550 ft at the crest of the 1950 flood. It may be noted from fig. 27 that s decreased considerably as the river rose and that a change probably occurred in borrow pit conditions during the 7-year interval between the 1943 and 1950 floods; values of s for 1950 are about 175 ft less than those for the 1943 high-water period at the same river stage. From the values of s plotted versus river stage in fig. 27 it is estimated that s might be as close as 500 ft at the Caruthersville site during the project flood.

165. Seepage exit. Values of x_3 were determined using the same piezometers as were used for estimating s . The average ground surface and tailwater assumed in the analysis was 268.0 at line A and 270.5 at line C. The resulting exit lengths are plotted vs corresponding river stages in fig. 27. At line A, where the top stratum consists of about 6.5 ft of clay silts, x_3 was about 240 ft at the crest of both the 1943 and 1950 high waters. At line C, where the top stratum consists essentially of about 7.5 ft of silty sand, x_3 was about 200 ft during the two floods. It is estimated that x_3 will be about 200 ft at the project flood at both piezometer lines A and C.

166. Thickness and permeability of substratum sands. The pervious foundation at Caruthersville consists of an upper stratum of fine sands about 10 ft thick and strata of medium to coarse sands about 100 ft thick

(plates 5-7). Permeabilities of these strata could be estimated only from laboratory permeability data, as grain-size data and seepage measurements were not available. Laboratory permeability tests on remolded samples from bailer boring K-1 (plate 6) indicate a k_f of about 80 x 10^{-4} cm per sec for the upper, fine sands, and 1150 x 10^{-4} cm per sec for the lower, coarse sands. On the basis of classification and pumping tests at other sites it is believed that the upper sands at this site have a k of about 100 x 10^{-4} cm per sec, and that the principal aquifer has a k of about 1500 x 10^{-4} cm per sec.

167. Thickness and permeability of top stratum. At line A the average top stratum landward of the berm toe is composed of about 6 ft of clay silts, except where ditches exist along Highway 84, underlain by 1 to 2 ft of sandy silt. On the basis of $z_{bL} = 6.5$ ft, k_{bL} was computed to be 20 x 10^{-4} cm per sec from formula 5, using $x_3 = 240$ ft as obtained from 1950 high-water data. At line C the top stratum (about 7.5 ft of silty sand) has an estimated k_{bL} of about 25 x 10^{-4} cm per sec. On the basis of these values it appears that during high water the clay silt top stratum at Caruthersville has a permeability almost equal to that of the silty sand top stratum.

168. Permeability ratio. The ratio of permeability k_f/k_{bL} of the foundation to that of the clay silt top stratum at line A is estimated to have been 75, and that of the top blanket of silty sand at line C to have been about 60 in 1950. Estimates of k_f/k_{bL} for the crest of the 1943 and project floods are given in table 4.

169. Seepage flow. Seepage passing beneath the levee at lines A and C at the crest of the 1950 flood, and for the project flood, was estimated using corresponding values for H , s , and x_3 for these floods. The seepage at the 1950 crest was about 250 gpm per 100 ft of levee at both lines A and C. Estimated Q_s at the project flood is about 600 gpm per 100 ft of levee. Q_s per H in 1943, 1950, and for the project flood is given in table 4. Because of the short length of piezometer lines A and C it is not possible to estimate accurately how much of the seepage passing beneath the levee in 1943 and 1950 actually rose to the surface. However, it is believed that approximately 50 per

cent of the seepage passing beneath the levee at the crest of the 1950 high water was emerging landward of the levee. During higher river stages when the ground storage will probably be filled farther landward of the levee, practically all of the seepage passing beneath the levee will rise to the surface. Thus it may be concluded that the Caruthersville site is subject to a high rate of natural seepage. (Although no actual seepage measurements were made at the site, it was reported that the seepage landward of the levee was very heavy during the 1950 high water.)

170. Landside substratum pressures. The hydrostatic pressures that developed at the toe of the berm at or near the crest of the 1943 and 1950 floods are shown on plates 12 and 13 (line D). Plots of readings of selected piezometers at or near the landside toe of the levee vs river stages are shown on plates 14 and 15, together with estimated substratum pressures for river stages up to the project flood. The head against the levee, the type and thickness of top stratum, and substratum pressures at each piezometer along the landside toe of the berm are given in table 5. From plates 9-11 it may be noted that maximum heads landward of the levee lagged only about one day behind the crest of the river. The lag of approximately five days (plates 14 and 15) in the development of excess head landward of the levee after the river reached the levee can be attributed to filling of the natural ground storage.

Table 5

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Caruthersville, Mo., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (277.4): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (287.2): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
D	1	264.7	---	9.3	5.0[e]	14.3	14.3	12.2	12.7	1.4	11	----	----	----	0.10	22.5	3.6[a]	16	>22.5
D	2	268.4	---	---	----	8.3[f]	8.3	7.1	9.0	1.1	12	----	----	----	0.13	18.8	2.1[d]	11	10.6
A-D	5 and 6	268.0	---	3.0[g]	1.5[g]	4.5[g]	3.8[g]	3.2	9.4	2.0	21	----	----	0.53	----	19.2	3.2[c]	17	10.8
C-D	9 and 10[h]	270.5	---	0.0	7.5	7.5	7.5	6.4	6.9	1.7	25	0.23	----	----	----	16.7	4.0[d]	24	10.0
D	11	269.2	---	2.1	1.2[e]	3.3	3.3	2.8	8.2	2.3	28	0.55	----	----	----	18.0	2.8[c]	16	9.3
D	12	270.5	---	0.0	8.2	8.2	8.2	7.0	6.9	1.8	26	----	----	----	0.22	16.7	4.5[d]	27	10.2
D	13	264.0	---	2.0	13.0	15.0[f]	15.0	12.8	13.4	5.6	42	----	----	----	0.37	22.2	12.8[c]	57	20.6

a, c, d See paragraph 143.
e Silt on top of clay.
f Stratified silt and clay.
g Thickness of top stratum beneath landside ditch.
h At edge of clay filled channel.

171. Plate 14 indicates that uplift pressures at piezometer 1 on line D probably will not be critical at the project flood because the estimated h_o (3.5 ft) for that flood will be 8.6 ft less than that required to cause sand boils or uplift on the basis of $i_c = 0.85$. Although the top stratum at piezometer 2, on the same basis, is thick enough to witnstand a hydrostatic head of 7.1 ft, it appears from the slope of the plotted data that h_o probably will not exceed a maximum of 1.5 to 2 ft, which probably will occur at a river height of about 11.5 ft on the levee, or 7 ft below the project flood. Although no sand boils were reported during 1950, they can be expected at a river head of 12 ft or more.

172. The computed h_c at the toe of the berm at line A is 3.2 ft based on the thickness of blanket at the bottom of the two ditches (see plates 5 and 14). However, the maximum h_o likely to develop at line A is only 2.0 ft, which may be expected by the time $H = 12$ ft. Sand boils are also likely to develop at this head. At piezometer line C, h_c = 6.4 ft; however, sand boils started in the vicinity of piezometers 9 and 10 with h_o equal to only 1.0 to 1.5 ft. Thus, the gradient required to cause sand boils in this area is considerably less than 0.85, and sand boils may be expected whenever the river is higher than 6.5 ft. At piezometer 11 ($h_c = 2.8$ ft) sand boils started with $i \cong 0.55$ and $H = 8.5$ ft. The computed maximum h_c, based on $i_c = 0.85$, and the estimated h_o at project flood stage are both equal to about 12.8 ft at piezometer 13. The head, h_o, will probable become equal to h_c at piezometer 13 when H = about 21 ft.

Evaluation of seepage problem and recommendations for control measures

173. Heads of about 8 and 14 ft on the levee during the 1950 and 1937 high waters, respectively, caused the formation of sand boils along the levee toe at the Caruthersville site except in the vicinity of piezometers 1 and 13. Substratum pressures that develop at piezometer 1 during a project flood will probably not be sufficiently critical to require control measures.

174. On the basis of piezometric data obtained during the 1943 and

1950 high waters, sand boils may be expected at flood stages 8 to 10 ft below a project flood stage. Higher stages will probably cause the formation of additional sand boils and increase the severity of the seepage. Sand boils may be expected to develop first between sta 26/10 and 26/17, where the top stratum is extremely thin (plate 7), and will probably also occur at relatively low river stages between sta 25/40 and 26/10. In the latter reach the severity will probably be greatest between sta 26/0 and about 26/10, because of proximity of the drainage ditch along the landside toe of the levee and the greater head.

175. At piezometer 13, sta 30/46+60, h_c can be expected to develop at a head 2 ft less than the project flood. However, in view of the limited information available concerning conditions in the vicinity of piezometer 13, no recommendation regarding control measures is made for this reach of levee.

176. Since the maximum H at the Caruthersville site is only about 18 ft and a somewhat smaller levee withstood a head of 14 ft in 1937, seepage beneath the levee is not considered particularly critical considering the added safety provided by the existing seepage berm. An important factor in this evaluation is the fact that the top stratum is relatively uniform in character and thickness so that there probably will be no high build-up of excess pressures or sufficient concentration of seepage to cause a blowout or serious underground piping. However, because of the close source of seepage in the riverside borrow pits and the perviousness of the foundation sands, heavy seepage and numerous sand boils can be expected when river stages exceed el 280.

177. In summary, the existing berm is considered somewhat narrow and a little thin in certain reaches; however, the only additional control measure recommended at the Caruthersville site is the construction of temporary abatis dikes in the riverside borrow pits to promote the collection of silt and growth of willows which will eventually lengthen the effective path of seepage.

Gammon, Arkansas

178. Gammon was selected as a site for study and installation of

piezometers because of its record of underseepage and sand boils. The site was also excellent for a study of the effect of borrow pits on seepage flow and substratum pressure, as the Mississippi River is some 2-1/2 miles from the levee and the area riverward of the borrow pits is blanketed with a fairly thick layer of clay for at least 2000 ft. The top stratum landward of the levee generally consists of relatively uniform clay 6 to 10 ft thick except in a drainage ditch along the levee where it is only 4 to 6 ft thick.

Description of site

179. The site is located approximately 10 levee miles north of West Memphis, Ark., between U. S. Highway 61 and the Mississippi River, and extends from levee sta 138/0 to 139/31. Plans of the site, river, borrow pits, surface geology, topography, and piezometers are shown on plates 16 and 17; plate 18 is an aerial mosaic of the site. The levee at Gammon has a net height of approximately 25 ft and a seepage berm 250 ft wide along its landside toe. Riverside borrow pits 5 to 10 ft deep and 500 to 600 ft wide, in which most of the top blanket has been removed, extend along most of the section of levee studied. A drainage ditch approximately 2 ft deep is located parallel to and about 150 ft landward of the seepage berm toe. River stages at Gammon can be estimated from the Memphis gage and the graph on plate 32.

180. History of underseepage. During the 1937 high water, when H = 20 ft, heavy underseepage was observed in landside borrow pits and in a sublevee basin between sta 138/0 and 139/30. Numerous sand boils ranging from pin size to 12 in. in diameter were observed in the sublevee area. The plan of the sublevee area as it was in 1937 is shown on plate 16. Locations and size of sand boils that occurred during the 1937 high water are shown on plate 17. Between the 1937 high water and the beginning of the underseepage investigation at Gammon the landside borrow pits were filled, and in 1946 the sublevee was replaced with a relatively wide, thin seepage berm. Typical sections of this berm are shown on plate 20.

181. During the 1950 high water (maximum H = 12.5 ft) heavy underseepage and approximately 40 sand boils, ranging in diameter from 3

to 12 in., were observed along the landside toe of the seepage berm between sta 138/5 to 139/10. The boils were active and some discharged an estimated 1/2 to 2 cu yd of sand; some discharged medium to coarse sand. Many of the boils were estimated to be flowing at 25 to 40 gpm. Medium to light seepage with some sand boils also was observed emerging through the berm. The specific locations of the 1950 underseepage and sand boils are shown on plate 17. The natural seepage Q_A emerging between the levee and the landside edge of the drainage ditch between sta 138/0 and the 138/22+75 was measured at the point indicated on plate 17 and equalled 56 gpm per 100-ft station for $H = 11.5$ ft, or approximately 5 gpm per ft H per 100-ft station.

182. Piezometer installation. In 1948 lines of piezometers were installed perpendicular to the levee at sta 138/10+50 (line C), 138/26+00 (line D), and 139/10+00 (line G), and at the toe of the levee at sta 138/0 and 138/45. The tips of all the piezometers were installed in clean sand immediately beneath the top stratum. Piezometer readings were obtained during the 1950 and 1951 high waters except at piezometers D-7 and F-15 in the borrow pit which were not read in 1950.

Geology of site and soil conditions

183. The general geology of the site is illustrated on plate 16. More detailed information regarding the type and thickness of top stratum materials is given by the colored hatching on plate 17. The site is located mainly in a relatively small area of point bar deposits bordered on the landside by river course 9 filled with relatively thick clay and silt deposits. On the riverside the site is also bordered by an old channel, also filled with relatively thick deposits of silt and clay (plates 16, 17, and 20). The point bar materials consist of a relatively uniform deposit of clay, with some minor amounts of silt, approximately 5 to 12 ft thick with a few intervening minor swale deposits (plates 17, 20, 22, and 23). These materials were deposited as the river gradually migrated from course 8 to the position marked by course 9. As a result, irregularities in thickness of the top stratum largely reflect the existence of subsurface ridges and swales, the axes of which generally parallel the alignment of the levee. Thin, predominantly clayey natural

levee deposits, which are difficult to distinguish from the underlying point bar clays, blanket the site.

184. Relation of underseepage to geology. The underseepage and sand boils that occurred during the 1937 high water can be attributed largely to riverside borrow pits excavated to sand and to the thinness of the top stratum landward of the levee, which was reduced in thickness by excavation for borrow for the sublevees.

185. The character and configuration of the top stratum together with the locations of seepage and sand boils that occurred during the 1950 high water are shown in detail on plate 17. An examination of this plate reveals that most of the sand boils were located in the narrow area between the toe of the present seepage berm and the thick clay deposits lying immediately landward of the levee, and in a shallow ditch where the natural thickness of the top stratum had been reduced by approximately 2 ft It is interesting to note again that most of the sand boils occurred opposite borrow areas riverward of the levee where the top stratum had been largely removed. It is also interesting to note that two sand boils occurred in the landside drainage ditch in the vicinity of the intersection of the present seepage berm toe and the edge of river course 9 (sta 138/6). Several active sand boils also were observed in this ditch at about sta 139/10 where an appreciably thicker clay blanket lies immediately landward of the berm toe and drainage ditch.

186. The primary factors affecting the severity and location of underseepage and sand boils at Gammon are open riverside borrow pits and the thinness of the natural landside top stratum, aggravated by the excavation of a drainage ditch immediately landward of the present seepage berm toe. The presence of thicker clay deposits immediately landward of the berm toe between sta 138/10 and 139/10 also contributes to the underseepage problem at this site.

187. Soil profiles and piezometer lines. The locations of piezometers and borings are shown in plan on plates 16 and 17. Soil profiles and piezometer lines, both perpendicular and parallel to the landside toe of the levee at Gammon, are shown on plates 19 to 23. The piezometer lines perpendicular to the levee were located at points where

underseepage had been the most severe (lines D and G) during the 1937 high water, and at a point considered to be critical (line C) where the clay filling in river course 9 intersects the levee toe at approximately sta 138/10. The locations of the piezometer tips are shown on the soil profiles.

188. Deep borings to Tertiary revealed a 140-ft-thick stratum of very pervious sands extending from the river to landward of the levee. In general, the natural top stratum immediately landward of the levee toe between sta 138/10 and 139/10 consists of a relatively uniform clay with a thickness of 6 to 10 ft, which has been reduced to only 4 to 6 ft along some reaches by the drainage ditch landward of the seepage berm toe.

Analysis of piezometric and seepage data

189. Piezometric and seepage data were obtained at Gammon during the 1950 high water (maximum H = about 13 ft); the piezometer readings are plotted together with river stages on plates 24 and 25. Piezometric gradients in the pervious substratum beneath the levee at piezometer lines C, D, and G are shown on plates 26-28 for selected river stages during the 1950 high water; the gradient at line D at about the crest of a high water in 1951 is also shown on plate 27. The hydrostatic head along the toe of the seepage berm as measured by piezometers along line B is shown on plate 29. From these plates, it can be seen that excess heads of about 2.5 to 5.5 ft developed at the toe of the berm from sta 138/0 to 139/10 at the 1950 crest. For the river stages experienced, little or no head existed above the top of the seepage berm. Local pressure relief along the toe of the seepage berm, as a result of seepage and boils, reduced the substratum pressures as indicated by the dotted lines on plates 26-27. It can also be seen that the gradients are relatively flat at distances greater than 300 ft landward from the berm toe. This is attributed in part to the ditch at the toe of the berm into which a major portion of the seepage entered, and in part to the clay-filled channel of course 9 landward of the levee which blocks the emergence of seepage beyond its near edge. The natural seepage emerging in an area about 250 ft wide landward of the levee was measured in 1950 at the point

shown on plate 17. These seepage measurements were used in estimating the permeability of the top stratum and the ratio k_f/k_{bL}. A summary of information pertaining to the site and analyses of piezometric and seepage data are given in table 6.

190. Source of seepage. Seepage may enter the pervious substratum at Gammon in the main channel of the Mississippi River and through the riverside borrow pits where most of the natural blanket has been removed (plates 17, 20, and 21).

191. Distances to the effective source of seepage were determined from readings of piezometers on lines C, D, and G for various river stages during the 1950 flood. The distances to the effective source were referred to a point 400 ft landward of the center line of the levee (about the toe of the seepage berm), because most of the seepage emerged landward of this point. The values of s obtained for various river stages during the 1950 high water for the different piezometer lines are shown in fig. 28 and plates 26-28. The data in fig. 28 show that seepage enters the sand substratum primarily through the borrow pits, even though some clay remains in the pits at piezometer lines C and G. The distance to the effective source of seepage at lines C and G was about 1100 ft. At line D, s was only about 600 ft. The short distance from the

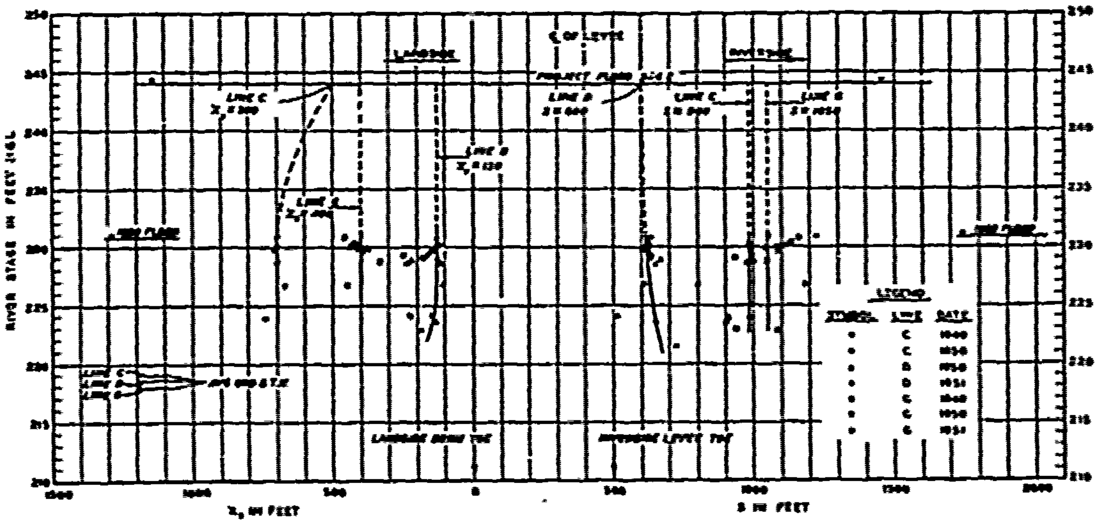

Fig. 28. Distances to effective seepage source and exit. Gammon, lines C, D, and G

Table 6

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Gammon, Ark., Site

Factor	Line C: 1950 Flood	Line C: Project Flood	Line D: 1950 Flood	Line D: Project Flood	Line G: 1950 Flood	Line G: Project Flood	Average Design Values: Sta 138/4 to 138/15	Sta 138/15 to 138/45	Sta 138/45 to 139/8	Sta 139/8 to 139/13	Sta 139/13 to 139/18	Sta 139/18 to 139/24
River stage (crest)	230.9	244.2	230.9	244.2	230.9	244.2	244.2	244.2	244.2	244.2	244.2	244.2
Average el of ground or tailwater	219.0	219.0	218.7	218.7	218.0	218.0	219.0	218.0	218.0	218.0	218.0	218.0
Head on levee (H)	11.9	25.2	12.2	25.5	12.9	26.2	25.2	26.2	26.2	26.2	26.2	26.2
Piezometers used in analysis	3, 4, & 5*	3, 4, & 5*	9 & 10	9 & 10	16 & 17	16 & 17	-----	-----	-----	-----	-----	-----
Riverside borrow pit, width	350	-----	560	-----	530	-----	350	450	450	450	450	450
Top stratum	0.4 ft clay	-----	0.2 ft clay 0.2 ft silt	-----	2.6 ft clay	-----	-----	-----	-----	-----	-----	-----
Average stratum	2 ft clay	-----	2 ft Si Cl	-----	4 ft clay	-----	2 ft clay	1 ft clay	3 ft clay	3 ft clay	3 ft clay	3 ft clay
Distance from riverside levee toe to river (L_1)	20,000	-----	20,000	-----	20,000	-----	20,000	20,000	20,000	20,000	20,000	20,000
Base width of levee (L_2)**	500	-----	500	-----	500	-----	500	500	500	500	500	500
Landward extent of top stratum (L_3)**	300†	-----	300††	-----	500	-----	{300‡ {300‡‡	300‡ 900‡‡	300‡ 2000‡‡	500‡ ∞‡‡	500‡ ∞‡‡	500‡ ∞‡‡
Distance to effective seepage source (s)**	900	900	610	600	1050	1050	1000	600	1000	1000	1000	1000
Effective length of riverside blanket (x_1)	400	400	110	100	550	550	500	100	500	500	500	500
Distance to effective seepage exit (x_3)**	700	500	130	130	400	400	{750‡ {1100‡‡	450‡ 500‡‡	650‡ 750‡‡	800‡ 870‡‡	950‡ 1000‡‡	600‡ 750‡‡
Effective thickness of sand substratum (d)	130	-----	135	-----	135	-----	130	135	135	135	135	135
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1000	-----	1000	-----	1000	-----	1000	1000	1000	1000	1000	1000
Laboratory permeability tests	-----	-----	750	-----	-----	-----	-----	-----	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	1200	-----	1200	-----	1200	-----	-----	-----	-----	-----	-----	-----
Seepage and piezometric data	850§	-----	850§	-----	-----	-----	-----	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Top stratum, type	Clay	-----	Clay	-----	Clay	-----	Clay	Clay & Silt	Clay	Clay	Clay	Clay
Effective thickness for seepage analysis (z_{bL})	7.5	-----	5.5	-----	8.5	-----	7.0	5.0	7.0	8.0	10.0	7.0
Critical thickness (z_t)	7.5	-----	4.0	-----	8.0	-----	5.0§§	4.0§§	6.0§§	8.0§§	10.0§§	6.0§§
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	5	7.5	40	40	10	10	-----	-----	-----	-----	-----	-----
Piezometric data and blanket formulas	5.3	7.4	44	44	9.5	9.5	-----	-----	-----	-----	-----	-----
Piezometric data and seepage measurements	5.2§	-----	5.0§	-----	-----	-----	-----	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	200	135	25	25	100	100	300‡ 600‡‡	200‡ 400‡‡	300‡ 600‡‡	300‡ 700‡‡	300‡ 800‡‡	300‡ 600‡‡
Blanket formula	200	135	23	23	106	106						
Natural seepage measurements	200	-----	200	-----	-----	-----	-----	-----	-----	-----	-----	-----
Natural seepage beneath levee												
Q_s, gpm/100 ft of levee (computed)	135	330	330	-----	175	360	-----	-----	-----	-----	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	11.3	13.1	27.0	-----	13.6	14.3	-----	-----	-----	-----	-----	-----
Q_s/H, gpm/100 ft levee between sta 138/0 and 138/22+75 and between levee and L.S. drainage ditch (measured)	6.0§	-----	6.0§	-----	-----	-----	-----	-----	-----	-----	-----	-----

* Piezometer 3 appeared sluggish and therefore piezometer 5 was also considered in evaluating s and x_3.
** Measured from approximate toe of berm 400 ft landward ℄ levee.
† Computed from gradient (20 February 1950, plate 26) and fig. 24 for blocked exit.
†† By inspection of hydraulic grade line for 20 February 1950, plate 27.
‡ For design of relief wells.
‡‡ For design of berm with drainage ditch being filled.
§ Assuming all measured seepage actually rose to surface between sta 138/5 and 138/22+75 in an area 250 ft wide between the levee and drainage ditch and using average H, M, and z_{bL} values at lines C and D.
§§ Average thickness of top stratum beneath ditch.

riverside toe to the point of seepage entry (150 ft) at line D is attributed to the almost completely exposed foundation sands in the riverside borrow pits. The clays in the borrow pit at lines C and G apparently force the point of seepage entry about 500 ft farther from the levee than that at line D. The values of s were approximately constant for all river stages at all piezometer lines. The distance to the source of seepage at the crest of the project flood was assumed to be about the same as observed in 1950.

192. Seepage exit. The distances from a line 400 ft landward of the levee center line to the effective seepage exit were determined at lines C, D, and G. The average ground surface and tailwater used in the analysis at lines C and D were el 219.0 and 218.7, respectively. At line G the tailwater was assumed at el 218.0 on the basis of the average ground surface (plate 22) and estimated water surface in the drainage ditch (plate 31). Values of x_3 vs corresponding river stage are plotted in fig. 28. At line C, where the top stratum consists of about 7.5 ft of clay, x_3 was equal to about 700 ft at the crest of the 1950 high water. At line D where the blanket consists of about 5.5 ft of clay, x_3 was about 150 ft. At line G, the clay top stratum is about 8.5 ft thick and x_3 was about 400 ft. It is believed that for the project flood x_3 would be about the same as estimated for the 1950 high water. The comparatively close exit at line D as compared to lines C and G is attributed to the thinner top stratum at line D.

193. Thickness and permeability of substratum sands. The pervious foundation at Gammon consists of an upper stratum of fine sands about 25 ft thick with underlying strata of medium to coarse sands about 125 to 130 ft thick (plates 19-23, and 32). Based on laboratory permeability tests on undisturbed and remolded samples an average permeability of about 75×10^{-4} cm per sec was obtained for the fine sands; laboratory permeability tests on the coarser sands indicated a k_f of 750×10^{-4} cm per sec. The effective thickness of the aquifer is considered to be about 135 ft. From the correlation between D_{10} and k_f, the permeability of the substratum would be 1200×10^{-4} cm per sec. The permeability of the substratum computed from the natural seepage measured on 7 February

1950 (56 gpm per 100-ft station) in the vicinity of piezometer line C, and the corresponding piezometric gradient at line C was 850×10^{-4} cm per sec. On the basis of these data, the permeability k_f of the sand substratum at Gammon is probably about 1000×10^{-4} cm per sec.

194. Thickness and permeability of top stratum. The thickness and permeability of the top stratum were estimated for lines C, D, and G and are given in table 6. The effective thickness of top stratum z_{bL} was based principally on an average of conditions near the berm toe where heavy seepage and sand boils occurred. The permeability of the clay top stratum was determined from the natural seepage measurements made in the vicinity of lines C and D and was determined from blanket formulas at all three lines. The best estimate of k_{bL} at the crest of the 1950 high water is given in table 6 and ranged from about 5 to 40×10^{-4} cm per sec. The high values of k_{bL} can be attributed to the relatively thin top stratum and the numerous sand boils which permitted high rates of seepage with relatively little head loss through the top stratum.

195. Permeability ratio. The ratio of the permeability of the foundation to that of the top stratum is estimated to have been about 200 and 100, respectively, at lines C and G, but only about 25 at line D at the crest of the 1950 high water. Estimates of k_f/k_{bL} for the crest of the project flood are given in table 6.

196. Seepage flow. Seepage passing beneath the levee at lines C, D, and G at the crest of the 1950 flood and for the project flood, was estimated using corresponding values of observed H , s , and x_3 for these floods. At the 1950 crest Q_s was estimated to be about 150 gpm per 100 ft of levee (Q_s/H = 12 gpm) at lines C and G and about 300 gpm (Q_s/H = 27 gpm) at line D. At project flood stage, Q_s would probably be about 300 and 700 gpm per 100 ft of levee at lines C and G, and line D, respectively. Q_A/H as determined from seepage measurements near line C was about 6 gpm (table 6), or about half of the estimated total seepage passing beneath the levee was emerging between the levee and the drainage ditch, a short distance landward. From these flows, it may be concluded that a high rate of natural seepage occurs at the Gammon site.

197. Landside substratum pressures. Landside substratum pressures

that developed at the berm toe near the crest of the 1950 flood are shown on plate 29. Readings of certain piezometers landward of the levee have been plotted vs corresponding river stages on plates 30 and 31. Estimated substratum pressures for river stages up to a project flood stage are also shown on these plates. The head on the levee, type and thickness of top strata, and substratum pressures at typical piezometers along the landside toe of the berm are given in table 7.

198. Piezometer A-1 was installed near the intersection of the landward levee toe and filled channel (course 9, plate 17). It appears that the hydrostatic head at piezometer A-1 at the project flood probably will not be more than h_c (based on $i_c = 0.85$) because h_o at the corresponding river stage is estimated to be 3.3 ft less than h_c. At piezometer B-2, an excess head of 3.6 ft developed with a tendency to remain essentially constant near the crest of the 1950 flood. Although h_c = 4.7 ft (for $i_c = 0.85$), the maximum head that can develop appears to be only 3.6 ft, which corresponds to a gradient of 0.62. Similarly an excess head of 3.3 ft developed at piezometer C-5 at a river height of 9 ft and remained constant until the flood crested. This excess head corresponds to a gradient of only 0.44. Active sand boils can be expected in the area bounded by piezometers B-2, C-5, and C-6 at the project

Table 7

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Gammon, Ark., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (230.9): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (244.2): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
K	A-1[e,f]	222.0	-----	7.0	5.0[g]	12.0	12.0	10.2	8.9	1.8	20	----	----	----	0.15	22.2	6.9[a]	31	>22.2
B	B-2	219.5	219.0[h]	5.5[i]	0.0	5.5[i]	5.5[i]	4.7	11.9[j]	3.6[j]	30	0.62	----	----	----	23.2	4.7[c]	19	14.7
C-B	C-5	219.5	219.0[h]	7.0[i]	2.0[i]	9.0[i]	7.5[i]	6.4	11.9[j]	3.3[j]	28	----	0.44	----	----	25.2	3.3[b]	13	9.0
C	C-6[k]	219.0	-----	8.5	0.0	8.5	8.5	7.2	11.9	3.4	29	----	----	----	0.40	25.2	---	--	----
D-B	D-10	218.5	218.5	4.0[i]	0.0	4.0[i]	4.0[i]	3.4	12.4[j]	2.5[j]	20	0.63	----	----	----	25.7	3.4[c]	13	16.5
b	E-14	218.0	218.0[h]	7.5[i]	0.0	[illegible]	7.5[i]	6.4	12.9[j]	4.3[j]	33	0.57	----	----	----	26.2	6.4[c]	24	19.1
G-B	G-18	218.[illegible]	218.0[h]	8.0[i]	0.0	8.0[i]	8.0[i]	6.8	12.9[j]	4.0[j]	31	0.48	----	----	----	26.2	6.8[c]	26	19.5

a, b, c See paragraph 143.
e Piezometer at intersection of levee slope and berm.
f Near edge of filled channel.
g Silt or top of clay.
h Elevation of water in drainage ditch.
i Thickness based on top stratum beneath drainage ditch near piezometer.
j Heads referred to water in drainage ditch.
k At edge of clay filled channel.

flood stage as it will create an H about 15 ft greater than that required to develop sand boils (see plate 30). Seepage will also be concentrated in this area because of the thick clay deposit immediately landward of the levee.

199. A river height of 11.5 ft created an excess head of 2.5 ft and sand boils near piezometer D-10; additional boils may be expected at river heights of more than 12 ft. Sand boils also developed in the vicinity of piezometers E-14 and G-18 at $H = 12$ ft; h_o at these piezometers was about 4.0 ft.

Evaluation of seepage problem and recommendations for control measures

200. River stages during the 1950 and 1937 floods created net heads of 12.5 and 20 ft, respectively, on the levee and caused considerable seepage and sand boils; sand boils began to develop at an H of 9 to 11 ft. The project flood will result in an H of about 25 ft. River stages in excess of the 1950 flood will increase the number and severity of boils without much increase in substratum pressure landward of the levee. The landside drainage ditch aggravates the seepage condition at the site. The reach between sta 138/6 and 139/25 is the most critical. Concentrations of seepage can be expected at sta 138/6 at the edge of old channel 9, and as a result of the landside drainage ditch. Based on readings of piezometer A-1 conditions do not appear critical beneath the filled channel. Analyses of the piezometric and seepage data at the Gammon site indicate that the existing berm is somewhat too narrow and about 2 ft too thin between sta 138/4 and 139/24. Additional control measures recommended are either a line of relief wells along the toe of the existing seepage berm or an enlargement and extension of the existing berm including backfilling of the drainage ditch. Another method of seepage control that may be applicable at Gammon is blanketing of the exposed borrow pits with clay.

Commerce, Mississippi

201. Commerce was originally selected as a site for investigation

because heavy underseepage and numerous sand boils occurred there during the 1937 flood. In general, the site is located on point bar deposits, the top of which consists primarily of 3 to 6 ft of silt overlain by 2 to 4 ft of clay. The top stratum landward of the levee is characterized by numerous ridges and swales.

Description of site

202. The site is located approximately 10 miles north of Tunica, Miss., where the levee is approximately 2200 to 4500 ft from the Mississippi River. The site extends from levee sta 22/40 to 24/4; however, most of the investigation was centered in that reach of levee from sta 22/45 to 23/30.

203. Plans of the site, river, borrow pits, surface geology, topography, and piezometers are shown on plates 33 and 34. Plate 35 is an aerial mosaic of the site and includes as an insertion an aerial view made prior to construction of the existing levee. The levee has a net height of approximately 22.5 ft. Riverside borrow pits 5 to 10 ft deep and 500 to 700 ft wide, in which much of the riverward top blanket has been removed (plate 34), exist along the site from sta 22/40 to 23/30. A fairly large old slough exists landward of the levee. River stages at the site can be estimated from the Memphis gage and the graph on plate 32.

204. History of underseepage. During the 1937 high water, an H of 21 ft caused heavy underseepage from the levee toe for a distance 1000 ft landward between sta 23/5 and 23/18. Several quite large sand boils developed and numerous boils required sacking (see plate 32). The levee also became saturated for 50 to 75 ft up the landside slope. Heavy underseepage was also observed from sta 23/18 to 25/3. Locations and approximate size of the 1937 sand boils are shown in detail on plate 34. In 1940, a landside seepage berm about 8 ft thick at the levee toe and 180 ft wide was constructed along the levee (plate 34). Typical sections of this berm are shown on plates 36-40.

205. During the 1950 high water, when H = 9 ft , much of the area landward of the levee between sta 22/23 and 23/35 was covered with water, but no sand boils were observed along the toe of the seepage berm.

Seepage flow between sta 22/24 and 23/36 as measured on 8 February 1950 with H = 8.2 ft amounted to approximately 60 gpm per 100-ft station, or approximately 7.5 gpm per 100-ft station per ft H . The seepage flow was measured at the 3-ft culvert shown on plate 33.

206. An experimental relief well system[45] with wells on 50-ft centers was installed between sta 22/42+00 and 23/26+32 along the landside toe of the seepage berm in December 1942 and January 1943. These wells were placed in operation during high river stages that occurred in May and June 1943. The average well flow from this system was 22 gpm for H = 7.4 ft , or approximately 6 gpm per foot H per 100 ft of levee. The estimated combined well flow and natural seepage per 100 ft of levee per ft H was 7.5 gpm. This value checks the natural seepage measured during the 1950 high water without any relief wells in operation. The well installation at Commerce was plugged in December 1943.

207. Piezometer installation. The piezometers were installed in 1942 except for 1 through 13 which were installed in 1944. Piezometers E-3-X and H-10-Y were removed in 1944. The piezometer installation consists of several lines (E, H, M, O, R, S) perpendicular to the levee at sta 23/2+75, 23/10+75, 23/20+25, 23/25+25, 23/30, and 24/3+40, respectively, with piezometers along the toe of the levee between sta 22/46+25 to 23/30 (plate 34). Piezometer line H extends from an old abandoned levee near the bank of the present river channel to a point almost 2 miles landward of the present levee. The tips of a number of piezometers on lines H and M were installed at different elevations in the pervious substratum to measure the difference in artesian pressure at different elevations in the sand stratum (plate 39). In general, most of the piezometer tips are in the top of the main sand stratum. Readings were obtained at Commerce during the high water of 1943 and 1950.

Geology of site and soil conditions

208. The general geology of the site is illustrated on plate 33. More detailed information regarding the type and thickness of top stratum materials is shown on plate 34. The principal area of investigation is located in point bar deposits formed while the river occupied course 14 (plate 33) and is bordered on the landside by an old slough, also a

remnant of course 14. The point bar deposits landward of the levee at Commerce have 2 to 6 ft of relief (plates 34 and 39).

209. Landward of the old slough, shown on plates 33 and 34, are two broad, flat, arcuate topographic lows (river courses 9 and 10) filled with 15 to 20 ft of clay which is covered with a natural levee deposit of sandy silt and lean clay, presumably laid down while the river occupied course 14 (plates 34 and 39). These latter deposits also exist immediately landward of the levee at piezometer line S (plates 34 and 40).

210. The alignment of point bar, sandy ridges, and intervening swales is unusual. Under the levee and for some distance riverward they trend at an angle of about 45° to the levee, whereas landward of the levee the alignment is roughly parallel to the levee. The point bar area appears to be blanketed by thin natural levee deposits; however, they are difficult to distinguish from the underlying fine-grained top stratum of the point bar materials.

211. The point bar deposits consist of alternating ridges of sandy silts and silty sands with intervening swales filled with clay (plates 38-40). The swales are usually relatively shallow, the clay being only 8 to 12 ft deep, and except in one or two instances the sandy ridges are blanketed with 2 to 8 ft of clay. The irregularities and thickness of the top stratum largely reflect subsurface ridges and swales.

212. Relation of underseepage to geology. Most of the sand boils occurring during the 1937 high water were located between sta 23/4 and 23/27 and between the then-existing landside levee toe and a clay-filled swale paralleling the levee toe at a distance of about 300 ft (see plates 34 and 38-40). This swale is 100 to 200 ft wide and thickens from about 6 ft at sta 23/10 to 35 ft at its junction with the old slough at sta 23/30. The ground surface in the swale is 2 to 5 ft lower than the ground on which the levee is constructed. The width and depth of this swale no doubt tend to concentrate the seepage between it and the toe of the levee where the substratum hydrostatic head is greatest. It is pointed out that the toe of the existing seepage berm is now only about 100 ft from the edge of the swale. While the seepage berm increases the

length of the path of seepage it will tend to further concentrate seepage between the swale and the toe of the berm. The probable predominating causes of seepage and sand boils in this reach of levee are: the close seepage entrance in the riverside borrow pits which have been generally excavated to sand; the great depth of very pervious substratum sands; and the thinness of the landside top stratum immediately landward of the levee toe (plate 37).

213. The natural levee deposits underlain by impervious clays, which form a mantle 20 to 30 ft thick landward of the old slough, constitute an effective block to underseepage landward of the slough. Consequently, most underseepage in the point bar area landward of the Commerce levee must come to the surface in the area between the toe of the levee and the thick clay deposit landward. Little seepage was observed along the reach of levee between sta 23/40 and 24/20 where the top stratum is 10 to 20 ft thick and the borrow pits expose very little of the underlying pervious sands.

214. Soil profiles and piezometer lines. The locations of piezometer and borings are shown in plan on plates 33 and 34. Soil profiles and piezometer lines, both perpendicular and parallel to the landside toe of the levee, are shown on plates 36 to 40. Three principal piezometer lines perpendicular to the levee, lines E, H, and M, are located in the area where underseepage was most severe during the 1937 high water. One line of piezometers S perpendicular to the levee, is located downstream of the point bar area where relatively little underseepage was observed during the 1937 high water.

215. The sediments that make up the top stratum in the point bar area are quite variable in thickness and in type, ranging from clays to silty sands. The top stratum immediately landward of the levee in the area of heaviest underseepage (sta 23/5 to 23/30) consists of a thin layer of clay 3 to 5 ft thick underlain by 3 to 8 ft of silty sand or sandy silt. Numerous clay-filled swales and sandy ridges exist immediately landward of this reach of levee. Downstream of the old slough at sta 23/40 the top stratum consists of a fairly uniform layer of impervious deposits approximately 10 to 18 ft thick.

216. The pervious substratum in the Commerce area consists of sands and gravelly sands. Generally the pervious stratum increases in coarseness with depth but coarse sands do exist to some extent near the top. Borings reveal only an occasional thin lens of fine-grained soils in the pervious substratum. Two borings made to Tertiary revealed that the substratum was very pervious and approximately 175 ft thick. Mechanical analyses of sand samples taken from the pervious substratum at Commerce are shown on plate 32.

Analysis of piezometric and seepage data

217. River stage and piezometer readings obtained during the 1943 and 1950 high waters are plotted on plates 41-46. At the crest of these high waters, H = 8 to 9 ft. During part of the 1943 high-water period a relief well system was in operation; however, the system was closed between 9 April and 6 June. An analysis of this relief well system has been reported in reference 45 and is not repeated herein. The analysis of data that follows pertains to conditions observed when the wells were closed, except that estimates of the permeability of the substratum were based in part on well flows corresponding to piezometric data obtained when the wells were flowing. No relief wells were in operation during the 1950 high water and therefore the following analyses of the 1943 and 1950 piezometric data are on a comparable basis.

218. Piezometric gradients in the pervious substratum beneath the levee at piezometer lines E, H, M, O, R, and S are shown on plates 47-56 for selected river stages during the 1943 and 1950 high waters. Gradients at short piezometer lines perpendicular to the levee (lines A, C, F, and J) are shown on plates 57 and 58, which also include the hydrostatic head along the toe of the levee as measured by line T piezometers. From these plates it can be seen that excess heads of about 1.5 to 3.5 ft developed at or landward of the berm toe from about sta 22/48 to 23/25 at the 1950 crest. (The hydrostatic head as measured at the toe of the seepage berm at line S by piezometer 3-X is believed lower than the head that probably existed because the tip of this piezometer was not set in clean sand.) Hydrostatic heads were above the ground surface from the toe of the levee

to the slough landward of the levee (see piezometer lines E, H, M, O, and R, plates 47-54). The hydrostatic grade line in the pervious substratum at line H flattened rapidly landward of the levee; however, as may be seen from plate 50, it still had some slope landward of the slough indicating a certain amount of seepage beyond the slough. As the hydrostatic head was below the ground surface, none of this seepage emerged above ground but flowed into ground storage landward of the slough. The highest h_o during the 1950 high water was about 3.5 ft (see lines E, H, and M, plates 48, 50, and 52, respectively).

219. A summary of information pertaining to the site and results of analyses of piezometric and seepage data are given in table 8.

220. Source of seepage. Seepage may enter the pervious aquifer through the bank of an old channel of the Mississippi River and through riverside borrow pits (see plates 34, 36-39).

221. Values of s determined from piezometer lines E, H, and M for various days during the 1943 and 1950 high-water periods are plotted in fig. 29. (Because of the uncertainty as to the accuracy of the readings obtained from piezometer 3-X on line S, the value of s at this line was not determined.) The values of s shown in fig. 29 and plates 48, 50, and 52 indicate that seepage enters the sand substratum primarily

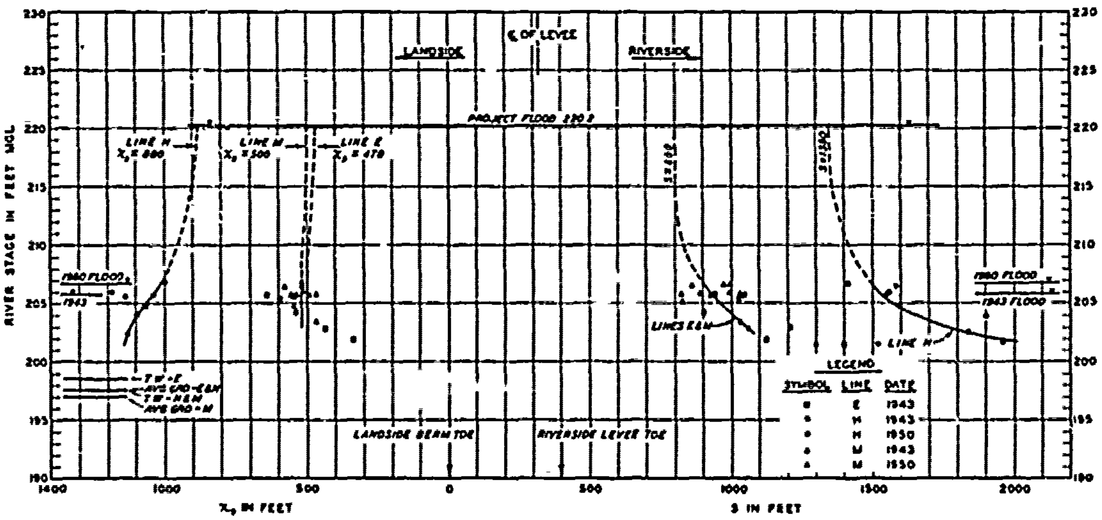

Fig. 29. Distances to effective seepage source and exit.
Commerce, lines E, H, and M

Table 8

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Commerce, Miss., Site

Factor	Line E 1943 Flood	Line E Project Flood	Line H 1943 Flood	Line H 1950 Flood	Line H Project Flood	Line M 1943 Flood	Line M 1950 Flood	Line M Project Flood	Line S Project Flood	Design Values Sta 22/45-23/25
River stage (crest)	205.7	220.2	205.7	206.7	220.2	205.7	206.7	220.2	220.2	220.2
Average el of ground or tailwater	198.5	198.5	197.5	197.5	197.5	197.5	197.5	197.5	197.5	197.5
Head on levee (H)	7.2	21.7	8.2	9.2	22.7	8.2	9.2	22.7	22.7	22.7
Piezometers used in analysis	2-X, 3-X*	-----	7-Y, 8-X, 11-Z	-----	-----	2-X, 4-Y	-----	-----	-----	-----
Riverside borrow pit, width, ft	750	-----	650	-----	-----	700	-----	-----	650	700
Top stratum	Sand	-----	0-6 ft Sd Si	-----	-----	5-15 ft Sd Si	-----	-----	0-6 ft clay 3 ft Si	-----
Average stratum	(None)	-----	3 ft Sd Si	-----	-----	8 ft Sd Si	-----	-----	5 ft clay	5 ft Sd Si
Distance from riverside levee toe to river (L_1)	2000	-----	1800	-----	-----	2000	-----	-----	2400	2000
Base width of levee (L_2)	400	-----	400	-----	-----	400	-----	-----	400	400
Landward extent of top stratum (L_3)	1500	-----	1400	-----	-----	950	-----	-----	*	1300
Distance to effective seepage source (s)	950	800	1525	1500	1350	940	910	800	Piezometer	800
Effective length of riverside blanket (x_1)	550	400	1125	1100	950	540	510	400	data too	400
Distance to effective seepage exit (x_3)	500	470	1050	1000	880	520	520	500	poor to be analyzed	600
Effective thickness of sand substratum (d)	165	-----	165	-----	-----	165	-----	-----	165	165
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1000	-----	1000	-----	-----	1000	-----	-----	-----	1000
Laboratory permeability tests**	750	-----	-----	-----	-----	-----	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	900	-----	-----	-----	-----	-----	-----	-----	-----	-----
Seepage and piezometric data	-----	-----	-----	875	-----	-----	470	-----	-----	-----
Field pumping tests	1000	-----	-----	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	865	-----	-----	-----	-----	-----	-----	-----
Top stratum, type	Cl & Sd Si	-----	Cl & Si Sd	-----	-----	Cl & Sd Si	-----	-----	Clay	Sd Si
Effective thickness for seepage analysis (z_{bL})	6.0	-----	7.0	-----	-----	7.0	-----	-----	8.0	7.0
Critical thickness (z_t)	6.0	-----	7.0	-----	-----	7.0	-----	-----	8.0	7.0
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	4.0	4.5	1.25	2.0	1.7	5.5	4.0	5.5	-----	-----
Piezometric data and blanket formulas	4.0	4.5	1.25	1.5	1.7	5.3	5.3	5.4	-----	-----
Piezometric data and seepage measurements	-----	-----	-----	3.2†	-----	-----	3.2†	-----	-----	-----
Permeability ratio (k_f/k_{bL})	250	220	800	500	580	190	250	180	-----	450
Blanket formula	250	220	800	720	580	190	190	185	-----	-----
Natural seepage measurements	-----	-----	-----	310	-----	-----	310	-----	-----	-----
Natural seepage beneath levee (computed)										
Q_s, gpm/100 ft of levee	122	420	80	91	257	139	160	440	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	17.0	19.5	9.8	9.9	11.3	16.9	17.4	19.3	-----	-----
Q_A/H, gpm/100 ft levee between sta 22/23 and 23/35 and between levee and L.S. slough (measured)	-----	-----	-----	6.6	-----	-----	-----	-----	-----	-----

* Head at E-3X was increased to obtain average head in sand substratum as described in paragraph 132.
** Based on D_{10} vs k_{LAB}, plate 24.
† Based on total measured seepage, average thickness of top stratum, and average excess head beneath top stratum for area landward of levee where observed seepage was emerging.

through the borrow pits where most of the natural impervious top stratum has been removed. It is pointed out that piezometers 1, 2, and 3 riverward of the levee on line H (plate 50) indicated hydrostatic heads almost equal to the river stage. The distance to the effective source of seepage at lines E, H, and M was about 950 ft, 1500 ft, and 925 ft, respectively, at the crest of both the 1943 and 1950 high waters (fig. 29 and table 8). Fig. 29 shows that s decreased considerably as the river rose. From the plotted values of s vs river stage in fig. 29, it is estimated that the source of seepage may be as close as 800 ft at lines E and M at the project flood stage, and 1350 ft at line H. At lines E and M the effective source of seepage entry thus would be only 400 ft from the riverside toe of the levee.

222. Seepage exit. Values of x_3 are plotted vs corresponding river stages in fig. 29 for both the 1943 and 1950 high waters. The average ground surface and tailwater used in the determination of x_3 are shown at the left in fig. 29. At piezometer lines E and M, where the top stratum consists of about 2 to 3 ft of clay underlain by about 3 to 6 ft of sandy silt, x_3 was about 500 ft at the crest of both the 1943 and 1950 high waters. At line H, x_3 was about 1000 ft. Although the top stratum immediately landward of the seepage berm at line H is similar to that at lines E and M, it is somewhat thicker and more impervious landward of the levee, which is probably the main reason for the greater x_3 at line H. It is estimated that at the project flood x_3 will be about 500 ft for piezometer lines E and M and about 900 ft for line H.

223. Thickness and permeability of substratum sands. The pervious foundation at Commerce consists of alternating strata of medium and coarse sands with some finer sand strata near the surface, some of which are also interspersed in the deeper and coarser strata (see plates 36 and 37). The pervious foundation has an effective thickness of about 165 ft. The permeability of the sand aquifer was estimated from laboratory permeability tests made on samples of sand obtained with a bailer sampler, correlation of D_{10} vs k_f as shown by fig. 17, seepage and piezometric data, relief well flow and related piezometric data, and

field pumping tests. The results of these determinations are summarized in table 8. The laboratory permeability tests indicated an over-all average k_f of about 750×10^{-4} cm per sec, whereas the other tests and determinations indicated a k_f of about 850 to 1000×10^{-4} cm per sec. The permeability as determined from the field pumping tests as described in Appendix C gave a k_f of 1000×10^{-4} cm per sec. As this value is considered to be the most reliable, it was used in the subsequent computations of seepage flow beneath the levee.

224. Thickness and permeability of top stratum. The average top stratum landward of the berm toe at Commerce is considered to have an effective thickness of about 6 to 7 ft; as may be seen from plates 38 and 39 the top stratum along the berm toe consists of about 2 or 3 ft of clay underlain by 3 to 6 ft of sandy silt or silty sand. However, it should be noted that the character and thickness of the top stratum landward of the levee vary considerably. On the basis of the top stratum thicknesses set forth in table 8, k_{bL} computed from piezometric data during the 1943 and 1950 high waters at lines E, H, and M was found to be about 2 to 5×10^{-4} cm per sec. The lower permeability of the top stratum at line H (see table 8) can probably be attributed to the greater extent and thickness of clay existing along this line. The permeability of the top stratum based on seepage measured landward of the levee during the 1950 high water, the average thickness of the top stratum, and excess head in the area landward of the levee where the observed seepage emerged was about 3×10^{-4} cm per sec.

225. Permeability ratio. The ratio of permeability of the foundation to that of the top stratum at piezometer lines E and M is estimated to be about 200 to 250; at line H the ratio is estimated to vary between 500 and 800. Estimates of k_f/k_{bL} for the crests of the 1943, 1950, and project floods are given in table 8.

226. Seepage flow. Seepage passing beneath the levee at lines E, H, and M at the crests of the 1943 and 1950 high waters, and for the project flood, was estimated using the corresponding values of H , s , and x_3 for these floods. The estimated Q_s at the 1943 and 1950 crests ranged from about 80 to 160 gpm per 100 ft of levee. Natural

seepage at a project flood stage would probably be about 250 to 450 gpm per 100 ft of levee. Q_s/H during the 1950 high water was estimated to range from about 10 to 17 gpm per 100 ft of levee; Q_A/H as determined from seepage measurements during this high water was about 7 gpm (see table 8). In comparing the above values of Q_s/H and Q_A/H, it is pointed out that Q_s/H as determined from piezometric data and foundation permeability characteristics represents the total seepage flow passing beneath the levee, whereas the value for Q_A/H includes only the seepage emerging between the levee and the slough landward of the levee. On the basis of the piezometric gradients observed at lines H and M during the 1950 high water, plates 50 and 52, respectively, it appears that approximately 80 to 90 per cent of the computed seepage Q_s was emerging landward of the levee during the 1950 flood. During higher river stages when the ground storage would probably be filled for greater distances landward of the levee, practically all of the seepage passing beneath the levee at Commerce may be expected to rise to the surface. From these data it may be concluded that the Commerce site is subject to a high rate of natural seepage.

227. Landside substratum pressures. The hydrostatic pressures that developed along the toe of the berm at or near the crest of the 1943 and 1950 floods are shown on plates 57 and 58 (line T). Plots of readings of selected piezometers at or near the landside toe of the levee vs river stages are shown on plates 59 and 60, which also show estimated substratum pressures for river stages up to the project flood stage. The head on the levee, top stratum characteristics, and substratum pressures at typical piezometers along the landside toe of the berm are given in table 9.

228. Plates 41-46 show that the maximum heads landward of the levee lagged only one or two days behind the crest of the river. The lag of approximately five days in development of excess heads landward of the levee after the river reached the levee can be attributed to filling of the natural ground storage landward of the levee as the river rose.

229. The data shown on plate 59 and in table 9 indicate that uplift pressures sufficient to cause sand boils can be expected to develop

Table 9

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Commerce, Miss., Site

Piez Line	Piez Number	Avg Ground at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (206.7): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med. Seepage	Light or No Seepage	Project Flood (220.2): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
A-T	A-1-X	201.0	—	3.5	5.0	8.5	6.0	5.1	5.7	1.0	18	—	—	0.20	—	19.2	2.6[d]	14	7.1
C-T	C-1-X	199.5	—	1.3	2.0	3.3	3.3	3.2	7.2	2.5	30	—	0.73	—	—	20.7	3.2[c]	15	7.8
E-T	E-3-X	199.0	—	3.0	3.0	6.0	4.5	3.8	6.7[e]	1.4[e]	21[e]	—	—	0.31	—	21.2	—	—	—
E-T	E-4-W	198.0	—	3.0	4.0	7.0	6.0	5.1	8.7	1.4	16	—	—	0.23	—	22.2	3.8[d]	17	—
E-T	E-5-X	199.0	—	3.0	5.5	8.5	7.5	6.4	7.7	2.7	35	—	—	0.36	—	21.2	6.0[d]	28	14.2
E-T	E-6-X	196.0	197.5	6.0	1.0	7.0	7.0	6.0	9.2	0.5	5	—	—	—	0.07	22.7	3.1[a]	14	>22.7
T	G-1-X	198.3	—	0.0	7.5	7.5	4.3	3.7	8.4	2.8	33	—	0.65	—	—	21.9	3.7[c]	17	11.0
H-T	H-9-X	197.8	—	2.5	6.5	9.0	3.8	3.2	8.9	2.2	25	—	0.58	—	—	22.4	3.2[c]	14	10.4
H-T	H-12-W	197.5	—	2.7	0.0	0.0	2.7	2.3	9.2	0.6	7	—	—	0.26	—	22.7	1.3[d]	6	13.2
J-T	J-1-X	199.4	—	5.0	4.0	9.0	5.8	4.9	7.3	1.7	23	—	—	0.29	—	20.8	4.9[c]	24	12.8
T	L-1-X	199.4	—	5.0	3.5	8.5	6.7	5.6	7.3	1.5	21	—	—	0.22	—	20.8	5.6[c]	27	14.8
M-T	M-3-X	198.5	—	3.0	8.0	11.0	5.8	4.9	8.2	1.2	15	—	—	0.21	—	21.7	4.8[d]	22	16.7
M-T	M-5-X	197.0	197.5	3.3	0.0	3.3	3.3	2.8	9.2	0.9	10	—	—	0.27	—	22.7	2.8[c]	12	16.2
T	N-1-X	198.0	—	0.0	9.0	9.0	7.0	5.9	8.7	1.3	15	—	—	0.19	—	22.2	4.1[a]	19	—
O-T	O-1-X	198.0	—	1.0	9.0	10.0	5.4	4.6	8.7	1.5	17	—	—	0.28	—	22.2	4.6[c]	21	—
O-T	O-2-X	196.8	197.5	2.0	6.5	8.5	6.0	5.1	9.2	1.5	16	—	—	0.25	—	22.7	4.2[d]	19	15.5
T	Q-1-X	198.0	—	1.5	10.3	11.8	3.6	3.1	8.7	1.6	18	—	—	0.44	—	22.2	3.1[c]	14	13.0
R-T	R-2-X	195.2	197.5	7.0	17.0	24.0	19.0	16.0	9.2	3.4	37	—	—	0.18	—	22.7	—	—	—
R-T	R-1-W	195.2	197.5	7.0	0.0	7.0	7.0	6.0	9.2	0.3	3	—	—	—	0.04	22.7	—	—	—
S-T	S-6-X	195.5	—	17.5	0.0	17.5	17.5	14.9	11.2	1.5	13	—	—	—	0.09	24.7	3.9[a]	16	>24.7

a, c, d See paragraph 141.

e 1943 flood.

along the toe of the existing seepage berm from sta 22/43 to 23/15 at river heights on the levee in excess of 8 to 12 ft. The lack of occurrence of sand boils during the 1950 high water may be attributed to the low head on the levee ($H = 8$ to 9 ft) and heavy uniform seepage which tended to prevent the development of any high uplift pressures ($h_o \leqq 1.0$ to 3.5 ft) ; see table 9. These values of h_o correspond to approximately 20 to 35 per cent of H . The maximum gradient through the top stratum observed during the 1950 high water ranged from about 0.6 to 0.7. In general, the gradients through the top stratum ranged from about 0.2 to 0.4. In summary, excess heads as high as 3 to 6 ft above ground and numerous sand boils may be expected to occur at the Commerce site from sta 22/43 to 23/30 when the river stage is higher than 10 to 15 ft. Excess heads greater than 3 to 6 ft are not expected to occur, as the top stratum can withstand only heads of this amount before it ruptures. H of more than 10 to 15 ft can be expected to increase the number and severity of the sand boils. In view of the fact that the project flood stage will create an H of approximately 22 ft, numerous and possibly severe sand boils will probably develop along this reach of levee at such a flood stage.

230. Exploration data downstream of sta 23/30 are inadequate to permit any accurate evaluation of conditions in this area. However, on the basis of rather questionable data from piezometer line S, conditions downstream of sta 23/30 apparently are not nearly as critical as those upstream of this point.

231. Hydrostatic head vs depth. Several piezometers were installed at various elevations within the pervious substratum and in the top stratum to determine variations in head at depth within the sand stratum and loss of head as the subsurface seepage rose to the surface. The piezometric head above the ground surface as measured by piezometers at different depths on lines H and M at about the crest of the 1950 high water (H = 8 to 9 ft) is plotted in fig. 30. This figure indicates that the hydrostatic head at a depth of approximately 40 ft below the top of

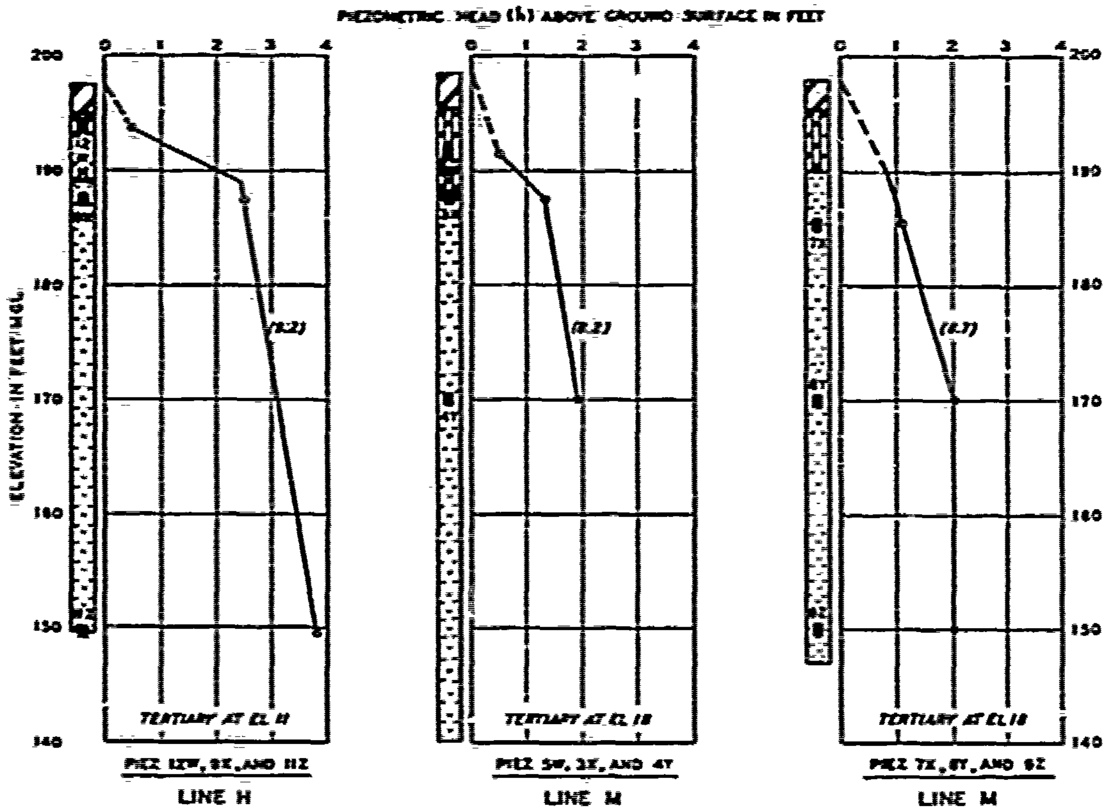

Fig. 30. Piezometric head above ground surface at various depths landward of levee, Commerce, Miss. (Note: figures in parentheses are river stage H above ground on 1 February 1950)

the pervious aquifer was approximately 1 to 1.5 ft greater than that immediately beneath the top stratum for the river stage experienced. It is also interesting to note that the rate of head loss up through the top stratum at the points of measurement was about as great through the silty sand and sandy silt material as through the thin upper layer of clay; however, this may well be expected where the clay stratum is very thin (only 2 to 4 ft thick) and is probably perforated by shrinkage cracks, root holes, crayfish holes, and other fissures.

Evaluation of seepage problem and
recommendations for control measures

232. Net heads of 8 to 9 ft on the levee during the 1950 high water did not cause the formation of sand boils along the levee at the Commerce site. However, from an analysis of the piezometric data and soil conditions at the site, it appears that uplift pressures sufficient to cause sand boils will probably occur when the river is higher than 10 to 15 ft on the levee (8 to 12 ft below the project flood stage). Although this levee withstood a head of approximately 20 ft during the 1937 high water, numerous sand boils occurred which required controlling. The seepage berm constructed in 1940 no doubt has improved the safety of the levee, in that it has practically eliminated any danger of sloughing of the landside slope as a result of seepage, and has forced the point of any subsurface piping farther from the levee proper. However, in the construction of this berm new borrow pits were opened riverward of the levee which probably reduced the distance to the effective source of seepage entry from that existing in 1937. Also, the berm and clay-filled swale and thicker top stratum immediately landward of the levee have created a condition that will cause concentration of seepage in a much narrower area from sta 23/0 to 23/30 than existed during the 1937 high water (see sections E, H, M, O, and R on plates 38-40).

233. In view of these conditions and because of the close source of seepage in the riverside borrow pits and the perviousness of the foundation sands, additional seepage control measures are recommended to further insure the safety of the levee at project flood stage. The existing berm has more than adequate thickness but is considered somewhat

narrow. Measures recommended are eithe. a line of relief wells along the toe of the seepage berm or widening of the berm.

Trotters 51, Mississippi

234. Trotters 51 was selected for intensive study because of the very serious underseepage and sand boils that occurred there during the 1937 h..gh water. Since 1937, a large seepage berm and sublevee basins have been constructed. The top stratum landward of the levee between levee sta 50/0 and 51/0 is extremely variable. Along the toe of the seepage berm the top stratum consists essentially of silt 5 to 20 ft thick; a thick, massive deposit of clay lies 100 to 300 ft landward of the berm toe. A reach of levee from sta 52/20 to 53/0 where the top stratum landward of the levee is quite thin (3 to 5 ft of clay with some silt and intervening sandy ridges and clay-filled swales) was also selected for study and installation of piezometers. This reach, although subject to seepage, had not been considered particularly critical.

Description of site

235. The site is located along the east bank levee of the Mississippi River approximately 5 miles west of Dundee, Miss., and extends from sta 50/0 to 53/20; however, most of the investigation was along that reach of levee from sta 50/15 to 51/0.

236. Plans of the site, river, borrow pits, surface geology, topography, and piezometers are shown on plates 61-63. Plate 64 is an aerial mosaic of the site taken before construction of the present levee and seepage berm. Plate 65 is an aerial mosaic taken after construction of the present levee, seepage berm, and sublevee basin. The levee is approximately 3500 ft from a former channel of the Mississippi River, and has a net height of approximately 25.5 ft. Riverside borrow pits 8 to 12 ft deep and 500 to 800 ft wide, in which most of the top blanket riverward of the levee has been removed, extend along most of the site. River stages at the site can be estimated from the Helena, Ark., gage and the graph on plate 83.

237. History of underseepage. During the 1937 high water, an H

of 21 ft caused very heavy underseepage and numerous sand boils 4 to 12 in. in diameter between sta 50/23 to 50/51. One 6-in. sand boil, when discovered at sta 50/40+15, was discharging water to a height of 2 ft above the ground surface. Opposite sta 50/42, a boil occurred about 200 ft from the levee toe east of a gravel road. This boil discharged considerable material as it moved across the road, causing the road to cave in to a depth of 15 ft to within 25 ft of the levee toe. Several sack sublevees were constructed around the active sand boils but other boils continued to break out beyond the limits of these sack levees. Finally, a large-sized sack sublevee was constructed from sta 50/23 to 50/51 which extended about 500 to 800 ft from the levee. More serious underseepage and sand boils occurred at this location than at any other in the Memphis District. At times, as many as 300 to 500 men were engaged in fighting underseepage. Approximately 160,000 sacks were used in constructing sublevees. Several sand boils also were observed between sta 51/38 and 51/48. The locations of the boils that occurred between sta 50/25 and 51/0 are shown on plate 62. Several photographs of conditions existing along this reach of levee during the 1937 high water are shown on plate 83.

238. In the summer of 1937 after the high water, permanent sublevees were constructed between sta 50/25 and 50/47, and a large landside seepage berm about 10 ft thick at the levee toe and 200 ft wide was constructed in 1938. This berm extends from sta 48/0 to 54/43+64. The location and extent of the permanent sublevee and seepage berm are shown on plates 61-63; typical sections of the sublevee and seepage berm are shown on plates 66-69.

239. During the 1950 high water (maximum $H = 11$ ft) 8 sand boils were observed between sta 50/5 and 50/40. The locations of these boils are shown on plate 62 by small solid dots. These boils were approximately 3 to 8 in. in diameter and were discharging considerable material at the crest of the high water. Numerous pin boils were observed in the landside drainage ditch between sta 51/37 and 51/53. Seepage water was observed in many of the fields and road ditches landward of the levee between sta 50/0 and 55/0. Considerable flow was emerging through a

culvert beneath the sublevee adjacent to the road at sta 50/44. This flow, which came from the lower two sublevee basins, was measured on 9 February 1950 when $H = 9.9$ ft, and was found to be approximately 240 gpm. This represents an average seepage flow from sta 50/33+70 to 50/47+27 of 17.7 gpm per 100-ft levee station, or Q_A/H = approximately 1.8 gpm per 100 ft of levee. However, it is pointed out that the measured seepage flow does not include all of the seepage passing beneath the levee along this reach, as there was still an appreciable flow of seepage landward of the sublevee basin as indicated by the slope of the hydraulic grade line at the landward side of the sublevee basin.

240. A relief well system[45] with wells on 50-ft centers was installed between sta 50/26+00 and 50/46+95 in January 1943. The wells were operated during high river stages that occurred in May-June 1943. The average well flow for this system was 16.6 gpm per 100 ft of levee for $H = 6.3$ ft, or an average flow of 5.3 gpm per 100-ft station per ft H. The well system was plugged in December 1943.

241. A partial cutoff 42 ft deep was constructed along the riverside toe of the levee between sta 51/6+38 to 51/20+70 in the fall of 1950. There has been no significant high water against this levee since installation of the partial cutoff.

242. Piezometer installation. In 1942 lines of piezometers were installed perpendicular to the levee at sta 50/36+50 and 52/22+00 with some piezometers along the toe of the seepage berm at sta 50/4, between sta 50/26+50 and 50/46+20, and between sta 52/48+00 and 53/3+40 (plates 61-63). Piezometer readings were obtained during the high water in 1943 and 1950.

Geology of site and soil conditions

243. The general geology of the site is illustrated on plate 61. The type and thickness of top stratum materials are illustrated in more detail on plates 62 and 63. The principal area of investigation (sta 50/0 to 53/15) is located in an area of point bar and channel deposits, which were formed while the river occupied courses 13 and 14, and is bordered on the landside by an old slough, a remnant of course 14 (plates 61 and 64).

244. From sta 48/19 to 53/10 the levee crosses an old sand bar deposit typified by ridge and swale topography underlain by deep deposits of pervious sand. Cross sections of the mapped areas show an undulated surface typical of ridge and swale topography. The maximum surface relief in this area is about 8 ft, but generally averages about 2 to 5 ft. (See plates 62-64, 66-67, and 69.) The ridges and swales formed during course 14 either parallel the levee or cross it at small angles.

245. A predominant feature of the surface geology immediately landward of the sublevee basin at Trotters 51 is a relatively thick (20 to 25 ft), narrow channel filling, the edge of which is approximately 300 ft from the landside toe of the levee as it existed in 1937 (plates 62 and 67). This channel filling does not exist at piezometer line H at sta 52/22 (plate 69). At this latter line of piezometers across course 14, the swale fillings consist of clay about 5 to 8 ft thick. In summary, the deposits landward of the levee from sta 50/0 to 53/10 consist of a very complicated sequence of channel fillings and ridge and swale deposits.

246. Clay deposits approximately 25 ft thick exist landward of former river courses 13-B and 14, and are overlain by natural levee deposits of sandy silt and fine sands 5 to 10 ft thick (see plates 61, 64, 66, and 69). The thickness and crevasse topography of these natural levee deposits, as vividly illustrated in the aerial mosaic on plate 64, indicate that the river must have occupied courses 13-B and 14 for many years.

247. Relation of underseepage to geology. The most severe underseepage and sand boils occurring during the 1937 high water were located between sta 50/23 and 50/51, and between the then-existing landside levee toe and the thick channel filling 300 ft landward, the bottom of which is filled with clay (plates 62 and 66). The top stratum between the levee toe and this filled channel consists of sandy silts and silty sands varying in thickness from 5 to 15 ft (see plate 62, sections C and E on plate 66, and sections F and U on plate 68). The presence of the deep clay- and silt-filled channel landward and parallel to the levee toe, and the silty ridge at the toe of the levee doubtless serve to localize

and concentrate underseepage along this reach of levee. Variation in thickness and the occurrence of occasional clayey seams in the sandy ridge at the levee toe are no doubt conducive to the concentration of seepage at this point (see plates 66 and 68). The clay-filled channel extends landward at piezometer line E until it joins a clay-filled slough, creating in effect a block to the emergence of underseepage over an area approximately 800 to 1000 ft wide landward of the levee (plates 62 and 67).

248. It is pointed out that the toe of the existing seepage berm is now only about 100 ft from the edge of the clay-filled channel. While the seepage berm increases the length of path of seepage and moves the point of any potential piping farther from the levee toe, it will in this instance tend to further concentrate seepage between the deep clay filling and toe of the berm. Other predominating causes of seepage and sand boils in this reach are the close effective seepage entrance in the riverside borrow pits, which have been excavated to sand, the great depth of pervious substratum sands, and the thinness and variability of the landside top stratum immediately landward of the levee toe.

249. Conditions similar to those in the vicinity of the sublevee basin described above exist along other reaches of the levee from sta 50/0 to 53/0. However, the particular combination of factors that causes underseepage to be so severe in the vicinity of the subleveed area does not exist to the same extent elsewhere. Either the top stratum is thicker landward of the levee or the riverside borrow pits have not uncovered the pervious sand foundation to the same extent, or the strata landward of the levee are more uniform, thereby allowing more uniform upward flow of seepage.

250. It would at first seem that a potentially critical underseepage area exists at about sta 53/0, where the clay-filled slough previously mentioned intersects the levee at a small angle (plate 63). The reason that no serious underseepage has been observed to date at this intersection is probably because the riverside borrow pits immediately landward of the levee in this area do not penetrate to the principal underlying pervious substratum, and also because a stratum of sandy

silts and clay lies under this particular point at a depth of approximately 20 ft (see sections J and L, plate 69).

251. Soil profiles and piezometer lines. The locations of piezometers and borings are shown in plan on plates 61-63. Soil profiles and piezometer lines, both perpendicular and parallel to the landside toe of the levee at Trotters 51, are shown on plates 66 to 69. One piezometer line (E) was located perpendicular to the levee in the area where underseepage was the most severe during the 1937 high water; another line (H) was located perpendicular to the levee in an area of sand bar deposits where no serious underseepage had occurred during the 1937 high water. Both piezometer lines E and H extend from near the riverbank for almost a mile landward of the levee. Additional piezometers were located at selected points along the landside toe of the present berm from sta 50/0 to 53/5. The tips of some piezometers, designated W , were installed in semipervious strata within the principal top stratum; the tips of the remaining piezometers, designated X , were installed in the upper part of the underlying pervious stratum beneath the top stratum.

252. The sediments that make up the top stratum in the sand bar area at Trotters 51 are quite variable, both in thickness and type, as may be noted on plates 62, 63, and 66-69. These sediments range from clay to silty sand. Most of the top stratum immediately landward of the levee from sta 50/0 to 53/10 consists of alternate layers of silty sand and sandy silt with occasional strata of clayey soils near the surface. Numerous clay-filled swales also exist along this reach of levee.

253. The clays and silty clays landward of the levee at Trotters 51 are usually found as fillings in swales or abandoned channels. Sandy silt and silty sand make up the greater part of the top stratum in the area and form the upper part of the ridges in the ridge and swale topography. They form the upper 5 to 15 ft of each ridge and grade upward into somewhat finer sediments. These sediments also frequently underlie swale fillings of clayey sediments and in places form part of old channel fillings.

254. At piezometers A-1 and B-1 the top stratum consists of about

4 ft of silty clay underlain by approximately 8 ft of sandy silt. The top stratum at the toe of the existing seepage berm from sta 50/15 to 51/20 consists of predominantly sandy silt and silty sand strata 5 to 15 ft thick interlaced with strata of sand and silty clay as depicted on section U, plate 68. The top stratum landward of the ridge of silt consists of a relatively thick, wide deposit of clay.

255. The top stratum along section H is representative of sandy ridge and swale deposits where both the ridges of sandy silts and clay-filled swales are relatively thin (see plate 69). The ridges usually are covered with a thin layer of clayey silt or silty clay 3 to 5 ft thick. The top stratum at the intersection of the old slough and levee at about sta 52/50 is depicted by sections J and L on plate 69. As may be noted, the top stratum consists of an upper stratum of silty sand approximately 4 ft thick underlain by about 10 to 15 ft of very fine sand, which in turn is underlain by a thin stratum of clays and silts about 3 ft thick.

256. The pervious substratum consists of medium to medium coarse sands with some gravel, and extends from the river to landward of the levee (see plate 67).

Analysis of piezometric and seepage data

257. River stage and piezometer readings obtained during the 1943 and 1950 high waters are plotted on plates 70-74. At the 1943 crest $H = 7$ to 9 ft; during the 1950 high water maximum $H = 9$ to 11 ft. The relief well system at the Trotters 51 site during the 1943 high water was kept closed until after the river crested. The analysis of piezometric data contained in this report pertains to conditions observed when the wells were closed, except that estimates of the permeability of the substratum were based, in part, on well flows corresponding to piezometric data obtained when the wells were flowing. No relief wells were in operation during the 1950 high water; and, therefore, the following analyses of the 1943 and 1950 piezometric data are on a comparable basis.

258. Piezometric gradients existing in the pervious substratum beneath the levee at piezometer lines E, H, and L for selected river

stages during the 1943 and 1950 high waters are shown on plates 75-80; the hydrostatic head along the toe of the levee (piezometer line U) is shown on plates 79-80.

259. Seepage and soil conditions vary considerably at the Trotters 51 site as illustrated by the surface geology depicted on plates 62 and 63. Accordingly in the following discussion, the piezometer lines are considered more or less representative of soil conditions along the following reaches:

Sta 50/0 to 50/28 -- piezometer lines A, B, and C
Sta 50/28 to 51/0 -- piezometer lines E and F
Sta 52/0 to 52/45 -- piezometer line H
Sta 52/45 to 53/5 -- piezometer lines J and L.

260. During the 1950 high water, h_o was approximately 7.5 ft above the elevation of the water in the sublevee basin (see line U, plate 80). Excess heads above the ground surface as high as 3 ft existed as far as 2500 ft landward of the levee at piezometer line E (see plate 76). At piezometer line H, h_o (immediately beneath the top stratum) was estimated to be approximately 2.5 ft at the 1950 crest; h_x as measured by deeper piezometers was approximately 3.5 to 2.5 ft as far landward as 700 ft (see plate 78).

261. A summary of information pertaining to the site, and the results of analyses of piezometric and seepage data subsequently discussed are given in table 10.

262. Source of seepage. Seepage may enter the pervious substratum at Trotters 51 from the Mississippi River, and through riverside borrow pits (see plates 62, 63, and 66-69). The worst seepage areas seem to be located directly opposite those borrow pits where all or practically all of the top stratum has been removed.

263. Values of s at piezometer lines E and H during the 1950 and 1943 high waters are plotted in fig. 31. These values indicate that seepage enters the sand substratum primarily through the riverside borrow pits where most of the natural impervious top stratum has been removed. It is pointed out that piezometer 1 riverward of the levee on line H (plate 78) indicated hydrostatic heads almost equal to the river stage. The distance to the effective source of seepage at

Table 10

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Trotters 51, Miss., Site

Factor	Line E: 1943 Flood	Line E: 1950 Flood	Line E: Project Flood	Line H: 1950 Flood	Line H: Project Flood	Design Values: Sta 50/25-51/0	Design Values: Sta 52/0-53/0
River stage (crest)	191.7	194.0	208.7	194.0	208.7	208.7	208.7
Average el of ground or tailwater	183.0	183.0	183.0	185.0	185.0	183.0	185.0
Head on levee (H)	8.7	11.0	25.7	9.0	23.7	25.7	23.7
Piezometers used in analysis	E-4X, -5X, -6X	E-3X, -4X, -5X, -6X	-----	H-2X, -4X	-----	-----	-----
Riverside borrow pit, width, ft	800	-----	-----	900	-----	850	850
Top stratum	0-5 ft Sa Si	-----	-----	0-8 ft Sa Si	-----	0-5 ft Sa Si	-----
Average stratum	3 ft Sa Si	-----	-----	1 ft Sa Si	-----	2 ft Sa Si	-----
Distance from riverside levee toe to river (L_1)	3400	-----	-----	3100	-----	3300	3100
Base width of levee (L_2)	450	-----	-----	450	-----	450	450
Landward extent of top stratum (L_3)	1600	-----	-----	1300	-----	1600	1300
Length of leaking blanket	500	-----	-----	-----	-----	500	-----
Distance to effective seepage source (s)	1150	1000	850	1100	900	850	550
Effective length of riverside blanket (x_1)	700	550	400	650	450	400	400
Distance to effective seepage exit (x_3)	1600	2500	750	735	650	2000	750
Effective thickness of sand substratum (d)	100	-----	-----	100	-----	100	100
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1000	-----	-----	1000	-----	1000	1000
Laboratory permeability tests	750	-----	-----	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	1000	-----	-----	-----	-----	-----	-----
Seepage and piezometric data	-----	835	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	1150	-----	-----	-----	-----	-----	-----
Top stratum, type	Cl & Sa Si	-----	-----	Cl & Sa Si	-----	Cl & Sa Si	Cl & Sa Si
Effective thickness for seepage analysis (z_{bL})	9.0	-----	-----	5.0	-----	8.0	5.0
Critical thickness (z_t)	9.0	-----	-----	5.0	-----	8.0	5.0
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	0.5	0.5	1.6	1.0	1.3	-----	-----
Piezometric data and blanket formulas	0.5	0.3	1.6	1.0	1.3	-----	-----
Piezometric data and seepage measurements	-----	0.7*	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	2000	2000	600	1000	770	1900	1000
Blanket formula	1980	3500	610	950	690	-----	-----
Natural seepage measurements	-----	1020	-----	-----	-----	-----	-----
Natural seepage beneath levee (computed)						-----	-----
Q_s, gpm/100 ft of levee	47.0	48.0	236	72.5	225	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	5.4	4.4	9.2	8.1	9.6	-----	-----
Q_A/H, gpm/100 ft of levee between sta 50/23 and 50/47+27 and between levee and L. S. sublevee (measured)	-----	1.6	-----	-----	-----	-----	-----

* Based on total measured seepage, average thickness of top stratum, and average excess head beneath top stratum for area landward of levee where observed seepage was emerging.

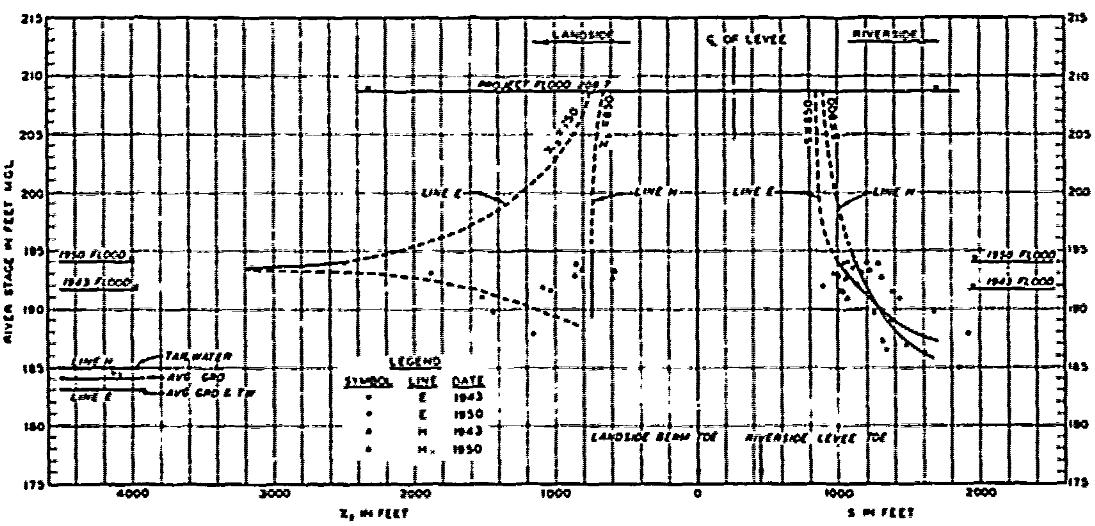

Fig. 31. Distances to effective seepage source and exit. Trotters 51, lines E and H

piezometer lines E and H was about 1000 and 1100 ft, respectively, at the crest of the 1950 high water (fig. 31 and table 10). It may be seen in fig. 31 that s decreased considerably as the river rose. From the plotted values of s vs river stage in fig. 31 it is estimated that s may be as close as 850 or 900 ft at lines E and H for a project flood stage. Thus the effective source of seepage entry would be only 400 or 500 ft from the riverside toe of the levee.

264. Seepage exit. Values of x_3 vs corresponding river stages are plotted in fig. 31 for both the 1943 and 1950 high waters. The very irregular topography, top stratum, and tailwater conditions landward of the levee at piezometer line E make a correct determination of the effective seepage exit difficult. On the assumption of an average ground and tailwater elevation of 183.0, it appears from fig. 31 that x_3 increased progressively during rising river stages during both the 1943 and 1950 high waters until H reached approximately 10 ft, at which stage x_3 equalled almost 3200 ft. At an H higher than 10 ft, the distance to the seepage exit decreased to a value of about 2500 ft. The distance to the effective seepage exit at project flood stage, based on the maximum h_o that can exist at line E, is about 750 ft. This phenomenon of an increasing x_3 followed by a decrease can possibly be explained as follows. At piezometer line E the top stratum immediately landward of the present seepage berm consists of a relatively narrow width of alternating strata of clay and silty soils and a wide width of fairly thick impervious soils, with a thinner top stratum approximately 1500 ft landward of the levee. The resistance of this thickness and type of top stratum to seepage rising to the surface is rather great. Accordingly, at relatively low river stages, values of x_3 probably reflect largely the resistance to seepage flow landward in filling ground-water storage. Then as the ground-water storage becomes filled, the resistance to seepage flow either landward or up through the top stratum is increased and results in an increase of x_3 . At a river stage of about 10 ft on the levee, several sand boils occurred in the sublevee basin immediately landward of the berm toe. Such boils, of course, provide a certain amount of natural relief close to the levee, which decreases

the distance to the over-all average seepage exit for the sand aquifer.

265. At the crest of the 1950 high water $x_3 \cong 700$ ft at piezometer line H (fig. 31 and table 10). It is expected that x_3 may decrease to approximately 650 ft at project flood stage. The much shorter x_3 at line H as compared to line E is attributed to a much thinner and more pervious top stratum at line H.

266. Thickness and permeability of substratum sands. The pervious substratum at the Trotters 51 site consists of medium to medium-coarse sands with some gravel, with a depth of approximately 100 ft. The gradations of typical foundation sands existing at this site are plotted on plate 83.

267. The permeability of the pervious substratum was estimated from laboratory permeability tests, correlation of D_{10} vs k_f as shown by fig. 17, analysis of seepage and related piezometric data, and well flow and related piezometric data. The results of these determinations are given in table 10. It may be noted that results obtained with the several methods used in arriving at the permeability of the substratum at this site checked reasonably well except those determined from laboratory tests which appear low. A value of $k_f = 1000 \times 10^{-4}$ cm per sec was selected as being the best estimate of the permeability of the sand aquifer in situ at the Trotters 51 site.

268. Thickness and permeability of top stratum. The top stratum landward of the levee at line E consists of alternating strata of clays, silts, and sands and is approximately 9 ft thick between the toe of the existing seepage berm and the sublevee. Beneath and landward of the sublevee a relatively thick, wide deposit of clay extends out a distance of approximately 1500 ft from the center line of the levee. Landward of this the top stratum for a distance of about 400 ft consists of about 10 ft of clay and clayey silt. In generalizing the top stratum at line E for the purpose of seepage analysis, the leaking blanket was assumed to have an effective thickness of 9 ft and a length of 500 ft, and an impervious block was assumed to exist at this distance from the toe of the seepage berm. On the basis of these assumptions the permeability of the top stratum as computed from blanket formulas and x_3 was 0.5 and

0.3×10^{-4} cm per sec at the crest of the 1943 and 1950 high waters, respectively. The permeability of the top stratum in the sublevee basin areas based on seepage measurements during the 1950 high water, the average thickness of top stratum, and the average head beneath the top stratum in the areas where the seepage was measured was 0.7×10^{-4} cm per sec. All factors considered, this is a relatively close check (see table 10).

269. The top stratum landward of the levee at line H consists of ridges of sandy silts and clay-filled swales; it was considered to have an average thickness of about 5 ft in the subsequent seepage analysis. On the basis of this assumption and using blanket formulas and measured values of x_3 , k_{bL} was computed to be about 1×10^{-4} cm per sec at the crest of the 1950 high water.

270. Permeability ratio. The ratio of permeability of the foundation to that of the top stratum was estimated on the basis of the above-described analyses and is shown in table 10 for both lines E and H. At the 1950 crest this ratio is estimated to have been 3500 for line E and about 1000 for line H.

271. Seepage flow. Seepage passing beneath the levee at piezometer lines E and H at the crests of the 1943 and 1950 high waters, and for the project flood, was estimated using corresponding values of H , s , and x_3 for these floods (see table 10). At the 1950 crest, Q_s passing beneath the levee at piezometer line E was estimated to be 48 gpm per 100 ft of levee, $Q_s/H \cong 4$ gpm per 100 ft of levee. Q_A/H as determined from seepage measurements near the crest of the 1950 high water in two of the sublevee basins was about 1.6 gpm. In comparing these values of Q_s/H and Q_A/H it is pointed out that Q_s/H as determined from piezometric data and foundation permeability characteristics represents the total seepage flow passing beneath the levee, whereas the value for Q_A/H includes only the seepage emerging in two sublevee basins immediately landward of the levee. On the basis of the piezometric grade line observed at line E at the 1950 crest it appears that approximately 70 to 80 per cent of the computed seepage was emerging between the levee and the slough approximately 1500 ft landward of the center line of the

levee. At project flood stage the natural seepage with no change from the 1950 borrow pit conditions is estimated at approximately 235 gpm per 100 ft of levee, $Q_s/H = 9$ gpm. Thus a fairly high rate of natural seepage occurs at line E at H higher than 10 to 15 ft.

272. The natural seepage passing beneath the levee at line H was estimated to be about 8 gpm per ft H per 100 ft of levee, or about 70 gpm at the crest of the 1950 high water (see table 10). The gradient line on plate 78 shows that practically all of this seepage emerges between the levee and the landward slough. From this same gradient line it appears that much of the natural seepage must have been emerging along the low ground between approximately 1000 ft from the center line of the levee and the slough. The fact that there was no indication of landward seepage beyond the slough can be explained by the existence of the very massive clay top stratum lying landward of the slough which prevents the emergence of seepage from the deep sand aquifer. No concentrated seepage or sand boils were reported during the 1950 high water; instead seepage was reported as emerging throughout the whole area between the levee and the landward slough (see plate 63). The estimated natural seepage beneath the levee at project flood stage is about 230 gpm per 100 ft of levee or $Q_s/H = 9.6$ gpm. This site may also be classified as one subject to a high rate of natural seepage at high flood stages. No measurements of natural seepage were made at line H during the 1950 high water.

273. Landside substratum pressures. Hydrostatic pressures which developed along the toe of the seepage berm (sta 50/28 to 51/0), piezometer line L, at the crests of the 1943 and 1950 high waters are shown on plates 79 and 80. Hydrostatic heads at and immediately landward of the seepage berm during these high waters are shown on the piezometric gradient graphs (plates 75-78). Readings of selected piezometers at or near the landside toe of the levee versus river stages are plotted on plates 81 and 82. The head on the levee, type and thickness of top stratum, and substratum pressures at typical piezometers along the landside toe of the seepage berm are given in table 11.

274. At piezometer line E, maximum heads landward of the levee lagged little, if any, behind the crest of the high water in 1943 and

Table 11

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Trotters 51, Miss., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, mGl	Est Tailwater el, ft mGl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (194.0): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (208.7): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
A-U	A-1-X	184.0^e	-----	6.0	7.3	13.3	8.5	7.2	10.0	4.2	42	----	0.49	----	----	24.7	7.2^c	29	15.6
B-U	B-1-W	184.0^e	-----	3.0	0.0	3.0	3.0	2.6	10.0	0.7	7	----	----	0.27	----	24.7	0.7^b	3	6.3
C-U	C-1-X	185.0	-----	----	----	18.0^f	18.0	15.3	9.0	5.5	61	----	----	0.31	----	23.7	15.3^c	64	20.4
		180.0^g	183.0	----	----	12.0	9.7	8.2	11.0	7.0	64	0.72	----	----	----	25.7	8.2^c	32	11.0
U	D-1-X	186.0	-----	----	5.0	9.0^h	9.0	7.7	8.0	4.5	56	----	0.50	----	----	22.7	7.7^c	34	12.5
		182.0^i	183.0	0.0	1.0	13.0^i	8.0	6.8	11.0	7.0	64	0.87	----	----	----	25.7	6.8^c	26	11.0
E-U	E-6-X	183.0	-----	----	----	10.0^f	10.0	8.5	11.0	7.4	67	0.74	----	----	----	25.7	8.5^c	33	11.0
E-U	E-7-W	183.0	-----	3.0	1.0 Sd	4.0	4.0	3.4	11.0	2.8	25	0.70	----	----	----	25.7	2.8^b	11	8.6
F-U	F-1-X	182.0	183.0	0.0	16.0	16.0	16.0	13.6	11.0	7.7	70	----	0.48	----	----	25.7	13.6^c	53	16.8
U	G-1-X	182.0	183.0	----	----	14.0^f	14.0	11.9	11.0	7.2	65	----	0.52	----	----	25.7	11.9^c	46	18.0
H	H-3-W	185.0	-----	2.5	4.5	7.0	7.0	6.0	9.0	3.0^j	33	----	----	0.43	----	23.7	6.0^c	25	11.4
		183.0^k	-----	0.5	4.0	4.5	4.5	3.8	11.0	2.7	25	----	0.60	----	----	25.7	3.8^c	15	9.5
J	J-1-X	182.5	183.0	3.0	17.0^l	20.0	15.0	12.8	11.0	3.8	35	----	----	0.25	----	25.7	12.8^c	50	23.0
L	L-2-X	185.0	-----	----	----	18.0^m	18.0	15.3	9.0	2.0^n	--	----	----	----	----	23.7	----	--	----
L	L-1-M	185.0	-----	----	4.0	4.0	4.0	3.4	9.0	0.0	0	----	----	----	0.0	23.7	3.4^c	14	15.4
		180.0^n	183.0	0.0	0.0	0.0	0.0	0.0	11.0	0.0	0	----	----	----	----	25.7	----	--	----

b, c See paragraph 143.
e Bottom of slough at elevation 182±.
f Alternating strata of clay, sand, and silt.
g Bottom of landside borrow pit.
h Top stratum 5 ft silty sand and 12 ft very fine sand.
i On slope in landside borrow pit at edge of clay top stratum. Top stratum 1 ft of silty sand and 12 ft of very fine sand.
j Estimated from H-4-X and fig. 26.
k Bottom of drainage ditch 60 ft landward.
l Silty sand and very fine sand.
m Thin clay stratum overlain by silty sand and very fine sand.
n Bottom of drainage ditch 150 ft landward.

1950 (see plates 70 and 72). The quick build-up of pressure with rising river stages can probably be attributed to the relatively thick top stratum that exists over most of the area landward of the levee at line E and to the fact that the bottom of this top stratum is below the normal water table. In fact, at line E during the 1943 high water, excess head above the ground surface appeared within three days after the river reached the levee.

275. Plate 70 shows a considerable lag in the crest of the piezometers on line H during the 1943 high water and the early stages of the 1950 high water. This may be attributed to filling of the natural ground-water storage landward of the levee at this line and emergence of seepage in the low ground landward of the levee. During the later stages of the 1950 high water, when the ground-water storage had become largely filled, the piezometers immediately landward of the levee on line H responded quickly to changes in river stage (see plate 73). Piezometers on lines J

and L exhibited even greater tendency to lag than those on line H (see plate 73).

276. Based on the data in table 11 and field observations, uplift pressure sufficient to cause sand boils in the sublevee basins may be expected at heads on the levee above 10 to 12 ft. These river stages created uplift pressures of from 5 to 7.5 ft in the sublevee basins; see table 11 and plate 80. These uplift pressures correspond to approximately 55 to 70 per cent H. The maximum excess head that can develop in the sublevee basins ranges from about 8 to 15 ft. In areas where sand boils were observed, i through the top stratum ranged from about 0.70 to 0.87. No sand boils occurred where i was less than these values. Heavy seepage was observed in a number of places where i ranged from about 0.5 to 0.6.

277. Since a project flood stage will create an H of approximately 25 ft, numerous active and possibly severe sand boils may be expected between sta 50/28 and 51/0. The reasons that the seepage conditions and sand boils will probably be more severe in this reach than immediately upstream are that seepage landward of the levee will be more concentrated because of the shorter distance between the seepage berm and massive clay filling immediately landward, the foundation sands are exposed immediately riverward of the levee along this reach, and the thickness of the top stratum has been reduced immediately landward of the levee for construction of the sublevees. Conditions similar to those found between sta 50/28 and 51/0 probably exist for a distance downstream of sta 51/0.

278. The fact that there is a maximum hydrostatic pressure that can develop beneath a given top stratum is again illustrated by the plot on plate 81 of piezometer readings for piezometer E-7-W on line E vs river stage. (Piezometer E-7-W is located in a sand stratum within the general top stratum at line E, see plate 76.) This plot illustrates how the substratum pressure increases with increasing river stage up to a certain amount equal to about the critical pressure for the weakest part of the top stratum in which sand boils were observed. In this instance the hydrostatic pressure at piezometer E-7-W remained essentially constant after the river stage became more than about 10 ft on the levee.

279. The head immediately beneath the top stratum at the toe of the seepage berm on line H was not observed during the 1950 high water; however, from readings of piezometer H-4-X and fig. 26 it was estimated to be about 3.0 ft above the ground surface. This is believed to be about the maximum h_o that can develop at this line because of the thinness of the top stratum and existence of a drainage ditch a short distance landward of the levee. Thus a river stage of 10 to 12 ft may cause sand boils along the toe of the seepage berm or in the drainage ditch landward of the levee.

280. Initially it was thought that a critical seepage condition might exist at about sta 53/0 (see plate 63). However, readings of piezometers along line L at about the crest of the high water in 1943 and 1950 (plates 79 and 80) show that for the flood stages experienced the hydrostatic head at the levee toe did not rise above ground surface except possibly for some excess head above the bottom of the drainage ditch at this point. The reason for the low hydrostatic pressures landward of the levee at piezometer lines J and L may be explained by reference to section J on plate 69, which shows that the upper stratum of very fine sands has a drainage outlet landward of the levee at an elevation 5 ft below natural ground surface at the seepage berm toe. Although no head above tailwater developed in the upper sand stratum, a head of almost 2 ft above estimated tailwater in the drainage ditch did develop in the deeper sand foundation at piezometer L-2-X.

Evaluation of seepage problem and recommendations for control measures

281. The seepage berm at this site has reinforced the stability of the landside portion of the levee and forced the location of boils farther from the levee proper. However, it also has reduced the area in which natural seepage can emerge between the levee and a massive body of clay immediately landward, and thus at times of high river stages will tend to increase substratum pressures and the number and activity of sand boils from sta 50/0 to 51/0, the limit of the area studied.

282. Sta 50/0 to 50/25. A net head of about 10 ft on the levee caused development of heavy seepage and small sand boils along this reach

during the 1950 high water. An analysis of piezometric data reveals that the 1950 high water was just sufficient to cause a few sand boils. Although few piezometric data are available for this reach of levee, it is believed that a project flood stage, which would be approximately 15 ft higher than the 1950 high water and 5 ft higher than the 1937 high water, will create an underseepage problem as bad as that occurring during the 1937 high water. On the basis of borings T-9 and T-12, the top stratum along the toe of the present seepage berm appears to be thick enough to withstand the uplift pressures that might develop. However, the top stratum apparently is thinner landward of the levee, as most of the sand boils occurring during 1950 were 100 to 150 ft landward of the berm toe (see plate 62). Additional borings should be made along this reach to better delineate the character and thickness of the top stratum landward of the levee; also additional control measures may be necessary along this reach of levee. In view of the fact that the sand boils that occurred during the 1950 high water were located some 100 to 150 ft landward of the existing seepage berm, it is believed that relief wells are a more applicable seepage control measure along this reach of levee than an extension of the present seepage berm. The existing seepage berm has more than adequate thickness.

283. Sta 50/25 to 51/0. Although the sublevee basins between sta 50/25 and 50/47 have added a measure of seepage control, in that the excess head can be reduced by impounding water in these basins, the seepage problem within the basins was aggravated by borrowing material within the basins to build the sublevees, which significantly decreased the thickness of top stratum within the basins proper. Also, since the 1937 high water, the distance to effective source of seepage entry into the pervious foundation probably has been decreased as a result of borrow operations riverward of the levee to construct the landside seepage berm. No information is available as to the distance to effective source of seepage entry during the 1937 high water. Even assuming water impounded in the sublevee basins to el 190, a project flood stage would create more than 8 ft of head above that necessary to cause formation of sand boils in the sublevee basin.

284. Critical uplift pressures and sand boils may be expected between sta 50/25 and 51/0 at river heights of more than 10 to 12 ft on the levee. Additional seepage control measures are indicated on the basis of possible river stages, field observations during previous high waters, soils data, and piezometric readings. Soil conditions along this reach of levee are such that an extension of the existing seepage berm is not considered practicable. The berm would have to be extended at least 150 to 300 ft and would have to be quite thick to insure that boils would not burst through it as a result of high pressures that would be created by tieing the berm into the thick clay deposits lying landward of the levee. Control measures recommended are the degrading of the existing sublevee basin so as to restore the ground surface to its original elevation and the installation of a line of relief wells.

285. Sta 52/0 to 52/45. The seepage berm along the reach of levee from sta 52/0 to 52/45 has more than adequate thickness but is not sufficiently wide to prevent development of critical uplift pressures landward of the berm. Critical uplift pressures may be expected to develop at river stages in excess of 10 to 12 ft and therefore some additional seepage control measures probably are indicated. This reach of levee is not considered as critical with respect to seepage as the reaches just discussed because the top stratum landward of the levee is uniformly thin which permits dispersion of natural seepage, and there is no thick stratum of clay closer than 600 to 800 ft to the levee to concentrate seepage. However, the drainage ditch about 80 ft landward of the toe of the seepage berm creates a potentially critical seepage condition. Recommended seepage control measures are either a line of relief wells along the toe of the existing berm or widening of the existing berm.

286. Sta 52/45 to 53/5. This reach of levee is particularly difficult to evaluate as regards seepage or adequacy of the existing seepage berm because of the topographical configuration, geology, and drainage ditch. No serious underseepage was reported during the 1937 high water, and river stages experienced during the 1950 high water were not great enough to cause the development of hydrostatic head above ground in either the upper fine sand strata or the deeper sands. However, the reach is

considered potentially critical with respect to seepage because of the 5-ft-deep drainage ditch and massive clay-filled slough a short distance landward of the levee. The seepage berm appears to have more than adequate thickness. However, its adequacy with regard to width cannot be evaluated on the basis of piezometer readings and river stages obtained since installation of the piezometers. The only additional seepage control measure recommended is the construction of a gated structure at the end of the drainage ditch to insure impoundment of water in the ditch up to natural ground elevation during high river stages.

Trotters 54, Mississippi

287. Trotters 54 was originally selected for investigation and installation of piezometers because of the very heavy underseepage and numerous sand boils that had occurred there during the 1937 flood. It was selected for installation of an experimental relief well system in 1950 for these reasons also, and because of the uniformity of soil conditions both riverward and landward of the levee, and the availability of results of previous geological and soil studies, as well as piezometric and seepage data. The top stratum landward of the levee consists of a medium thick (8 to 9 ft) clay blanket except where the thickness has been reduced by a landside drainage ditch 2 to 3 ft deep.

Description of site

288. The site is located along the Mississippi River levee across from Helena, Ark., just downstream of the Trotters 51 site. The Trotters 54 site as discussed herein extends from sta 53/25 to 54/28; however, most of the investigation was centered around that reach of levee from sta 53/36 to 54/28 (plates 61 and 84).

289. Plans of the site, borrow pits, surface geology, topography, and piezometers are shown on plates 61 and 84. Plate 85 is an aerial mosaic of the site and river taken after construction of the present levee and seepage berm. The locations of certain piezometers at the lower end of the Trotters 51 site are also shown on this mosaic. The extent of the relief well system at Trotters 54 and the locations of all piezometers

and borings are also shown on plate 85. The levee is approximately 2700 ft from the present course of the Mississippi River. Most of the top stratum riverward of the levee has been removed by riverside borrow pits 5 to 10 ft deep and 800 to 1000 ft wide. River stages can be estimated from the Helena, Ark., gage and the graph on plate 83.

290. History of underseepage. During the 1937 high water, an H of 23.7 ft caused very heavy underseepage from the levee toe for a distance 1000 ft landward between sta 53/34 and 54/26. About 300 to 500 relatively small sand boils occurred in this area.

291. In 1938 a large landside seepage berm about 11 ft thick at the levee toe and 200 ft wide was constructed. The location and extent of the seepage berm are shown on plates 61 and 84; typical sections of the berm are shown on plates 86-89.

292. During the 1950 high water (maximum H = 13.5 ft) heavy underseepage and numerous sand boils occurred in a drainage ditch immediately landward and parallel to the toe of the berm between sta 53/36 and 54/34. Most of these boils were 3 to 6 in. in diameter and were moving a small amount of sand when observed. The heaviest concentration of sand boils occurred between sta 53/52 to 54/10 where approximately 75 sand boils were found. Some of these boils were as large as 10 in. in diameter; 12 sand boils were counted in a landside drainage ditch between sta 54/10 and 54/13. Numerous small pin boils were also observed between the toe of the berm and the drainage ditch. Seepage rising to the surface immediately landward of the berm toe between sta 53/48+15 to 54/10+25 was measured on 8 February 1950 when H = 13.5 ft. The average rate of seepage along this reach of levee was 55 gpm per 100-ft station, or approximately 4 gpm per ft H per 100 ft of levee. The measured seepage flow does not represent the total seepage passing beneath the levee at this point, as there was still some flow of seepage landward of the drainage ditch as indicated by the slope of the hydraulic grade line immediately landward of the ditch.

293. A relief well system, with 2-1/2-in.-ID wells on 75-ft centers, was installed between sta 53/31+84 and 54/34+00 about 50 ft landward from the landside toe of the seepage berm in 1943.[45] These wells flowed during

high river stages that occurred in May-June 1943. The average well flow was 25.5 gpm for H = 9.4 ft, or a flow of 3.6 gpm per ft H per 100-ft station. The well system was plugged in December 1943.

294. An experimental relief well system, with 6-in.-ID wells on 50-ft centers, was installed between sta 53/51+55 and 54/8+25 along the landside toe of the seepage berm in 1950 (see Appendix D). These wells were operated during high river stages in 1951 and 1952. The average well flow was 75 gpm for H = 7.5 ft, or 20 gpm per ft H per 100-ft station.

295. Piezometer installations. Piezometers were installed in 1942, 1943, 1948, and 1950. The installation consists essentially of a line M of piezometers perpendicular to the levee at sta 54/1, with numerous piezometers along the toe of the seepage berm and in the well line. Piezometer readings were obtained during the high water in 1943, 1950, 1951, and 1952.

Geology of site and soil conditions

296. The general geology of the site is illustrated on plate 61. The type and thickness of top stratum materials are illustrated in more detail on plate 84. The levee at the site crosses an old river channel laid down during course 12, which extends from approximately levee sta 53/37 to 54/18. The sediments within this area consist of impervious top stratum and a thick pervious substratum extending to a depth of about 130 ft (see plates 86-89). From about sta 53/16 to 53/51 the top stratum consists of a fairly uniform clay layer about 15 to 20 ft thick, overlain by natural levee deposits 4 to 7 ft thick. From sta 53/51 to 54/18 the top stratum thins to a relatively uniform clay stratum only 10 ft thick, which extends along the toe of the levee. The top stratum thickens somewhat landward of the levee. A stratum of extremely uniform fine and very fine sand about 35 ft thick lies immediately beneath the top stratum of clays and silts; uniform medium to coarse sands approximately 90 ft thick lie below the fine sands. At lower depths a considerable amount of gravel is interspersed in the coarse sand. Tertiary materials underlie the sand substratum. This condition of an upper stratum of relatively impervious clays and silts underlain by a stratum of fine sands, in turn underlain

by a massive stratum of coarser sands and gravels is typical of many sites along the Lower Mississippi River levees, except that the stratum of very fine sands at Trotters 54 is somewhat thicker than usual.

297. Relation of underseepage to geology. As illustrated on plate 84, the character and thickness of the top stratum are relatively uniform except for a thicker deposit of clay at the upstream end of the site. Seepage was less severe during both the 1937 and 1950 high waters in the area covered by the thicker clay. The somewhat thicker top stratum landward of the levee toe at the center of the site no doubt exerts a certain concentrating influence on seepage between the levee toe and the thicker top stratum. The shallow drainage ditch immediately landward of the seepage berm toe at the site has caused most of the sand boils observed to date to form on the bottom and sides of this ditch. The rather severe seepage conditions at the Trotters 54 site may be attributed principally to the riverward borrow pits in which the top stratum has been removed down to the underlying clean pervious sands and which provide a ready entrance for seepage into the pervious foundation. The lesser seepage upstream of sta 53/45 is probably a result of both the riverside borrow pits not penetrating the top blanket and the top stratum landward of the levee being appreciably thicker.

298. Soil profiles and piezometer lines. The locations of piezometers and borings are shown in plan on plate 84. Soil profiles and piezometer lines both perpendicular and parallel to the landside toe of the levee at Trotters 54 are shown on plates 86-89. One perpendicular piezometer line M is located in the area where underseepage was most severe during the 1937 high water, and it extends from near the river to a distance of 3200 ft landward of the levee. Additional piezometer lines (N, Q, R, and O) were located perpendicular to the levee to provide a check on the distance to the effective source of seepage entry and seepage exit at the site. Numerous piezometers were located along the landside toe of the present seepage berm (line T) and in the line of relief wells (line U). The tips of most of the piezometers were installed in the upper part of the pervious substratum a short distance below the top blanket; however, as shown on plate 86, a number of tips were installed

at the bottom of the very fine sand stratum.

299. The thickness of the top stratum silts and clays at the site ranges from 9 to 11 ft except at the bottom of the drainage ditch where the thickness is only about 6.5 ft (plates 86-89).

300. The principal seepage-carrying stratum of medium and coarse sands at this site is quite pervious and averages about 85 to 90 ft in thickness depending on the thickness of the upper fine sand stratum and irregularity of the top of the underlying Tertiary formation.

Analysis of piezometric and seepage data

301. River stages and piezometer readings observed at the Trotters 54 site during the 1943, 1950, 1951, and 1952 high waters are plotted on plates 90-96. A relief well system (1943) was in operation during a part of the 1943 high-water period. An analysis of the 1943 relief well system at Trotters 54 has been reported previously[45] and is not repeated herein. After the high water of 1950 a new relief well system was installed at this site, and was in operation during the 1951 and 1952 high waters. The design and construction of this system and its operation in 1951 and 1952 have been reported in detail in reference 51; its design and performance are summarized in Appendix D. The following analysis pertains to conditions observed when the wells were closed, except that estimates of the permeability of the substratum were based in part on well flows corresponding to piezometric data obtained when the wells were flowing. The maximum heads created on the levee at Trotters 54 during the 1943, 1950, 1951, and 1952 high waters with the well systems closed were 9.2, 13.8, 5.5, and 8.8 ft, respectively.

302. Piezometric gradients in the pervious substratum beneath and landward of the levee at piezometer line M are shown on plates 97 and 98 for selected river stages and relief-well operating conditions during the 1950, 1951, and 1952 high waters. Gradients at piezometer lines R and 4 during the 1951 and 1952 high waters are shown on plate 99. The hydrostatic head along the toe of the seepage berm, line T piezometers, is shown on plate 100 for the 1951 and 1952 high waters. (Piezometer line T was not in existence during the 1950 high water.) From plate 97, it

may be noted that h_o at piezometer line M was approximately 5.0 to 5.5 ft above the ground surface at the crest of the 1950 high water. During this high water, excess heads of 2 to 4 ft existed as far as 2500 ft landward of the levee. Plates 97 and 98 show that the hydrostatic grade line in the pervious substratum at line M flattened rapidly landward of the levee. Thus most of the seepage passing beneath the levee, after the ground-water storage becomes filled, apparently rises to the surface between the levee and a point approximately 2000 ft landward.

303. A summary of information pertaining to the site and results of analysis of piezometric and seepage data are given in table 12.

304. Source of seepage. Seepage can enter the pervious substratum at Trotters 54 from the main channel of the Mississippi River and through riverside borrow pits.

305. Values of s are plotted for lines M, Q, and R in fig. 32. The values at the crest of the high waters are given in table 12. The values of s shown in fig. 32 and as depicted graphically on the piezometric gradients, plates 97-99, indicate that the seepage enters the sand substratum primarily through the riverside borrow pits where most of the natural impervious top stratum has been removed. It is pointed out that piezometers M-I-X, M-1, and M-8 riverward of the levee on line M indicated

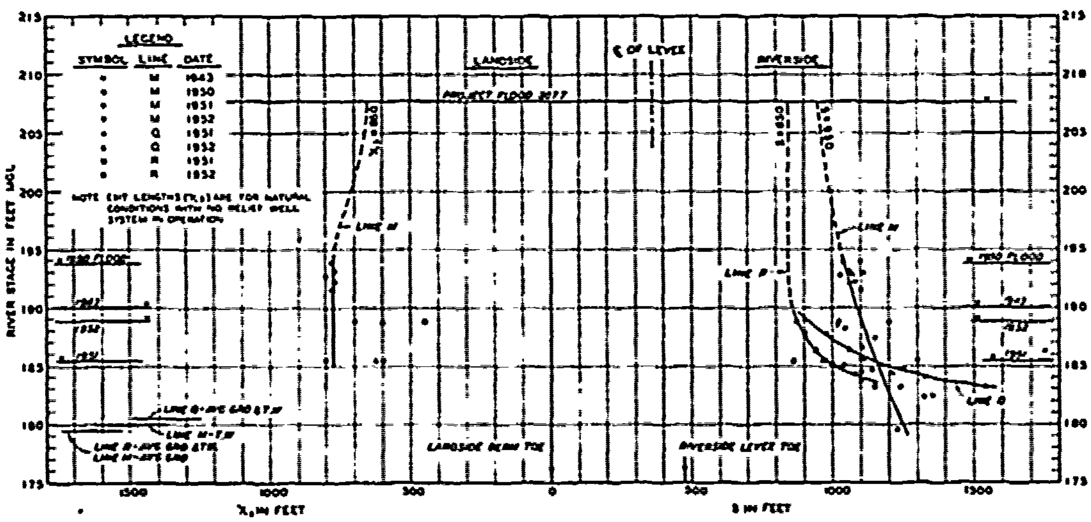

Fig. 32. Distances to effective seepage source and exit.
Trotters 54, lines M, Q, and R

Table 12

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Trotters 54, Miss., Site

Factor	Line M					Line P		Line Q		Line R			Line S		
	1943 Flood	1950 Flood	1951 Flood	1952 Flood	Project Flood	1951 Flood	1952 Flood	1951 Flood	1952 Flood	1951 Flood	1952 Flood	Project Flood	1951 Flood	1952 Flood	Design Values
River stage (crest)	189.2*	193.8	185.5*	188.8*	207.7	185.5*	188.8*	185.5*	188.8*	185.5*	188.8*	207.7*	185.5*	188.8*	207.7
Average el of ground or tailwater	180.0	180.0	180.0	180.0	180.0	180.5	180.0	180.5	180.5	179.5	179.5	179.5	180.0	180.0	180.0
Head on levee (H)	9.2	13.8	5.5	8.8	27.7	5.0	8.3	5.0	8.3	6.0	9.3	28.2	5.5	8.8	27.7
Piezometers used in analysis	M-2X, 2, 3, 4	-----	-----	-----	-----	P-1, -2	-----	Q-1, -4	-----	R-5, -7, -8	-----	-----	S-1, -2	-----	-----
Riverside borrow pit, width	900	-----	-----	-----	-----	900	-----	900	-----	900	-----	-----	900	-----	900
Top stratum	0-2 ft clay	-----	-----	-----	-----	10-15 ft clay		0-2 ft clay	-----	0-2 ft clay &	2-5 ft Sl Sa	-----	0-2 ft clay	-----	-----
Average stratum	1 ft clay	-----	-----	-----	-----	12 ft clay		1 ft clay	-----	3 ft clay	-----	-----	1 ft clay	-----	-----
Distance from riverside levee toe to river (L_1)	2500	-----	-----	-----	-----	3000	-----	2800	-----	2500	-----	-----	2500	-----	2500
Base width of levee (L_2)	475	475	-----	-----	-----	475	-----	475	-----	475	-----	-----	475	-----	475
Landward extent of top stratum (L_3)	∞	-----	-----	-----	-----	∞	-----	∞	-----	∞	-----	-----	∞	-----	∞
Distance to effective seepage source (s)	1100	1050	1150	1100	950	1375	1100	1150	900	975	875	850	1225	975	900
Effective length of riverside blanket (x_1)	625	575	675	625	475	900	625	675	425	500	400	375	750	500	425
Distance to effective seepage exit (x_3)	770	770	770	770	630	-----	790**	625†	600**	600†	450**	-----	800†	670**	750
Effective thickness of sand substratum (d)	90	-----	-----	-----	-----	90	-----	90	-----	90	-----	-----	90	-----	90
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1750	-----	-----	-----	-----	1250	-----	1250	-----	1250	-----	-----	1250	-----	1250
Laboratory permeability tests	400	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	1250	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Seepage and piezometric data	-----	1250	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Field pumping tests	-----	1180	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	1250	-----	1500	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Top stratum, type	Clay	-----	-----	-----	-----	Clay	-----	Clay	-----	Clay	-----	-----	Clay	-----	Clay
Effective thickness for seepage analysis (z_{bL})	9.0	-----	-----	-----	-----	16.0	-----	8.0	-----	9.0	-----	-----	8.0	-----	9.0
Critical thickness (z_c)	6.5	-----	-----	-----	-----	14.0	-----	5.5	-----	6.5	-----	-----	6.5	-----	6.5
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	1.7	1.4	1.7	1.7	2.4	-----	2.9	2.3	2.5	2.8	5.0	-----	1.4	2.4	-----
Piezometric data and blanket formulas	1.7	1.7	1.7	1.7	2.4	-----	2.9	2.3	2.5	2.8	5.0	-----	1.4	1.4	-----
Piezometric data and seepage measurements	-----	1.0††	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	730	730	730	730	520	-----	435	540	500	445	250	-----	900	515	700
Blanket formula	732	732	732	732	521	-----	435	542	500	445	250	-----	890	515	-----
Natural seepage measurements	-----	1250	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
Natural seepage beneath levee (computed)															
Q_s, gpm/100 ft of levee	81.4	125	47.4	78	286	-----	72.6	46.5	91.5	63.0	116	-----	45.0	92.0	-----
Q_s/H, gpm/ft of head/100 ft of levee	8.9	9.1	8.6	8.9	10.3	-----	8.8	9.3	11.0	10.5	12.5	-----	8.2	10.5	-----
Q_A/H, gpm/100 ft levee between sta 53/43+15 and 54/10+25 and between levee and L.S. drainage ditch (measured)	-----	4.0	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----

* Maximum river stage with wells closed.
** 4 April 1952, wells closed.
† 10 March 1951, wells closed.
†† Based on total measured seepage, average thickness of top stratum, and average excess head beneath top stratum for area landward of levee where observed seepage was emerging.

hydrostatic heads almost equal to the river stage. Values of s ranged from about 900 to 1100 ft at the crest of both the 1950 and 1952 high waters. It may be seen in fig. 32 that s decreased considerably as the river rose. From the values of s plotted vs river stage in fig. 32, it is estimated that s may be as close as 850 to 950 ft at the project flood stage if borrow pit conditions are not altered. Thus, the effective source of seepage entry would be only about 500 ft from the riverside toe of the levee at such a flood stage.

306. Seepage exit. Values of x_3 are plotted vs corresponding river stages in fig. 32 for the 1943, 1950, 1951, and 1952 high waters. A line of best fit has been drawn through the readings for line M in fig. 32. At this line, where the top stratum for a distance of approximately 500 ft landward of the seepage berm toe consists of about 9 ft of clay, x_3 was about 770 ft at the crest of the above-mentioned high waters. At piezometer lines P, Q, R, and S, x_3 at nearly the 1952 crest ranged from about 500 to 800 ft. In considering the distance to the effective seepage exit at this site, it is pointed out that much of the natural seepage passing beneath the levee rises to the surface between the levee and the 2-ft-deep drainage ditch 100 ft landward of the seepage berm. Also the clay blanket immediately landward of the seepage berm and in the bottom of the drainage ditch has been perforated previously by sand boils and therefore does not perform as a uniform or continuous layer of clay.

307. On the basis of extrapolation of the data plotted in fig. 32 and the computed maximum h_o that can exist beneath the top stratum landward of the levee at piezometer line M, x_3 would probably be about 650 ft at the project flood stage without the relief well system in operation. The rather sharp break in the seepage exit line at a river stage of 195 in fig. 32 is predicted on the basis that this stage will cause sand boils that will provide additional outlets for seepage, thereby reducing the distance to the effective seepage exit.

308. Thickness and permeability of substratum sands. As previously stated, the pervious foundation at Trotters 54 consists of an upper stratum of fine to very fine sands, approximately 20 to 40 ft thick with a k of approximately 50×10^{-4} cm per sec, underlain by the principal

seepage-carrying stratum consisting of medium to coarse sands about 80 to 90 ft thick. The gradation of typical foundation sands at the Trotters site is plotted in fig. D3, Appendix D. Photographs of two undisturbed samples of the upper foundation sands are shown on plate 83.

309. The permeability of the pervious substratum was estimated from laboratory permeability tests, correlation of D_{10} vs in-situ permeability as shown by fig. 17, seepage and piezometric data, field pumping tests, and well flow and piezometric data, and the results for line M are included in table 12. Reasonably good checks were obtained for the several methods used except for the k estimated from laboratory tests. The principal aquifer is considered to have an in-situ permeability of about 1250×10^{-4} cm per sec.

310. Thickness and permeability of top stratum. The top stratum landward of the levee has been described under "Geology of site and soil conditions." The effective thickness of top stratum used in the seepage analyses made at the various piezometer lines is given in table 12. The critical thickness of top stratum as regards uplift considerations and the development of sand boils is that beneath the drainage ditch previously referred to, and ranges from about 5.5 to 6.5 ft. The permeability of the clay top stratum as computed from blanket formulas and piezometric data obtained at the crest of the various high waters and the effective thicknesses shown in table 12 ranged from about 2 to 5×10^{-4} cm per sec. The permeability of a 100-ft-wide strip including the critical drainage ditch area was computed from measured seepage and average excess head beneath the area. By this method and from these data the permeability of the top stratum in this strip was found to be 1×10^{-4} cm per sec. The fact that the permeability of the clay blanket is very much greater than would normally be associated with such material is attributed to the existence of numerous crayfish holes, sand boils, and other fissures in the top stratum through which most of the seepage was emerging (see fig. 10 and photographs of undisturbed samples on plate 83).

311. Permeability ratio. Estimates of the ratio of permeability of the foundation to that of the top stratum along the several piezometer lines at the high water crests are shown in table 12. In general,

this ratio ranged from about 500 to 700.

312. Seepage flow. Seepage passing beneath the levee at the crests of the high waters, and for the project flood, was estimated using corresponding measured values of H, s, and x_3 for these floods (see table 12). At the 1950 crest Q_s at piezometer line M was estimated to be 125 gpm per 100 ft of levee. Q_s/H ranged from about 8 to 12 gpm per 100 ft of levee for the various piezometer lines and high waters. Q_A/H as determined from seepage measurements during the 1950 crest between levee sta 53/48+15 and 54/10+25, and between the levee and landside drainage ditch, was 4.0 gpm. In comparing the above values of Q_s/H and Q_A/H, it is pointed out that the Q_s/H as determined from piezometric data and foundation permeability characteristics represents the total seepage flow passing beneath the levee, whereas the value for Q_A/H includes only the seepage emerging between the levee and the [illegible]side edge of the drainage ditch. On the basis of the piezometric gradients observed at line M during the 1950, 1951, and 1952 high waters (plates 97 and 98) it appears that approximately 60 to 80% of the computed Q_s seepage was emerging within a short distance landward of the levee at the crests of these high waters, and that probably 90% of Q_s was emerging landward of the levee. At project flood stage the natural seepage with no change in borrow pit conditions from those existing since 1943 is estimated at approximately 300 gpm per 100 ft of levee, or $Q_s/H = 10$ gpm. From these data it may be concluded that the Trotters 54 site is subject to a high rate of seepage at significant flood stages. Flow from the 1950 relief well system when operating on 50- and 100-ft centers averaged about 16 gpm per ft H per 100 ft of levee during the 1951 and 1952 high waters (see Appendix D). No observable natural seepage rose to the surface landward of the well system when the well system was in operation. The head between wells, with the well system operating on 50- and 100-ft centers, was about 8 and 15 per cent H, respectively.

313. Landside substratum pressures. Hydrostatic heads at and immediately landward of the seepage berm during the high waters since 1943 are shown on plates 97-99. Hydrostatic pressures that developed along the toe of the berm at or near the 1951 and 1952 crests are shown on

plate 100, line T. Readings of selected piezometers at or near the landside toe of the levee vs river stages are plotted on plate 101. Also shown are estimated piezometer readings for river stages up to the project flood. The head on the levee, type and thickness of top stratum, and substratum pressures at certain typical piezometers along the landside toe of the berm are given in table 13. Plates 91-96 show that the maximum heads landward of the levee lagged little if any behind the crest of the river. During the 1950 high water there was an estimated lag of approximately 5 to 7 days in the development of excess heads landward of the levee after the river reached the levee. This is attributed to filling of the natural ground storage landward of the levee as the river rose. The well system at Trotters 54 began to operate during the 1951 and 1952 high waters with a head of only about 1 to 2 ft on the system. Thus the

Table 13

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Trotters 54, Miss., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c ($0.85\ z_t$) ft	Crest of 1950 Flood (193.8): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (207.7): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
N-T	P-2	181.0	-----	16.0	0.0	16.0	16.0	13.6	7.8	2.6[f]	36	----	----	----	0.18	26.7	---	--	---
		179.3[f]	179.5	14.3	0.0	14.3	14.3	12.2	9.3	4.3[e]	46	----	----	0.30	----	28.2	---	--	---
T	T-1	181.0	-----	9.5	3.0	12.5	9.8	8.3	7.8	1.7[e]	22	----	----	----	0.17	26.7	---	--	---
		178.9[f]	179.2	7.4	3.0	10.4	8.8	7.5	9.6	3.5[e]	36	----	----	0.40	----	28.5	---	--	---
T	T-2	181.0	-----	10.0	1.0	11.0	10.2	8.7	7.8	---	--	----	----	----	----	26.7	---	--	---
		178.6[f]	179.0	7.7	1.0	8.7	7.9	6.7	9.8	---	--	----	----	----	----	28.7	---	--	---
Q-T	Q-2	181.0	-----	7.5	0.5	8.0	7.7	6.5	4.8	0.6[g]	13	----	----	----	0.06	26.7	---	--	---
		178.5[f]	178.8	5.0	0.5	5.5	5.7	4.8	7.0	2.8[g]	40	----	0.49	----	----	[illegible]	---	--	---
Q	Q-4	181.0	-----	8.0	0.0	8.0	8.0	6.8	4.8	0.6[g]	13	----	----	----	0.08	26.7	---	--	---
		178.5[f]	178.8	5.5	0.0	5.5	5.5	4.7	7.0	2.8[g]	40	----	0.51	----	----	28.9	---	--	---
M-T	M-4-W	180.0	-----	9.0	0.0	9.0	9.0	7.7	13.5	3.0	22	----	----	0.33	----	27.7	3.0[b]	11	6.7
		177.5[f]	178.0	6.5	0.0	6.5	6.5	5.5	15.8	5.0	32	0.77	----	----	----	29.7	5.0[b]	25	8.7
M	M-14	179.5	-----	8.5	0.0	8.5	8.5	7.2	9.3	1.2[e]	13	----	----	----	0.14	[illegible]	---	--	---
		-----	-----	----	---	----	----	----	----	---	--	----	----	---	----	----	---	--	---
M	M-5-W	178.0	-----	11.4	3.6	15.0	11.5	9.8	15.8	5.0	32	----	----	0.44	----	29.7	[illegible][b]	33	---
		-----	-----	----	---	----	----	----	----	---	--	----	----	----	----	----	---	--	---
T	T-9	180.0	-----	10.0	0.0	10.0	10.0	8.5	5.8	1.5[g]	26	----	----	----	0.15	27.7	---	--	---
		177.5[f]	178.0	7.5	0.0	7.5	7.5	6.4	7.8	3.5[f]	45	----	0.47	----	----	29.7	---	--	---
R-T	R-6	180.0	-----	9.0	0.0	9.0	9.0	7.7	8.8	2.1[e]	24	----	----	----	0.23	27.7	---	--	---
		177.5[f]	178.0	6.5	0.0	6.5	6.5	5.5	13.8	4.1[e]	33	0.63	----	----	----	29.7	---	--	---
R	R-8	179.5	-----	9.0	1.5	10.5	9.0	7.5	9.3	2.1[e]	23	----	----	----	0.23	[illegible]	---	--	---
		177.5[f]	178.0	7.0	1.5	8.5	[illegible]	6.1	10.8	3.6[e]	33	0.50	----	----	----	29.7	---	--	---
T	T-11	180.5	-----	9.8	0.0	9.8	9.8	8.3	8.3	1.3[e]	16	----	----	----	0.13	[illegible]	---	--	---
		177.5[f]	178.0	6.8	0.0	6.8	6.8	5.8	13.8	3.8[e]	35	0.65	----	----	----	29.7	---	--	---
O-T	S-2	180.5	-----	9.5	0.0	9.5	9.5	8.1	8.3	2.3[e]	28	----	----	0.24	----	[illegible]	---	--	---
		177.5[f]	178.0	6.5	0.0	6.5	6.5	[illegible]	15.8	4.8[e]	45	0.74	----	----	----	29.7	---	--	---

[b] See paragraph 143.

[e] 1952 piezometer readings with flood stage = 188.8.

[f] Bottom of drainage ditch 100 ft landward of berm toe.

[g] 1951 piezometer readings with flood stage = 185.8

No relief well system in operation.

lag in development of excess heads landward of a levee during rising river stages depends to a marked extent upon previous river stages and ground-water storage conditions beneath and landward of the levee.

314. From the data shown on plate 101 and in table 13, and from field observations, uplift pressures sufficient to cause sand boils immediately landward of the levee and in the drainage ditch along the toe of the existing seepage berm may be expected between sta 53/50 and 54/20 at flood stages higher than 7 to 9 ft without the relief well system in operation. For these river stages, uplift pressures of 2 to 3 ft were observed at the toe of the seepage berm (see table 13 and plates 100 and 101). These pressures correspond to approximately 25 to 35% H . The net hydrostatic head above the water in the drainage ditch approximately 100 ft landward of the seepage berm toe ranged from about 4 to 5 ft. In the drainage ditch, where most of the sand boils occurred in 1950-1952, i through the top stratum ranged from about 0.5 to 0.8 (table 13) compared to a theoretical i_c of 0.8 for clays. The maximum gradient through the top stratum where no sand boils were observed was about 0.5. However, rather heavy seepage was observed in these areas.

315. In summary, it appears that excess heads as high as 3 to 5 ft and numerous sand boils may be expected at Trotters 54 site when H is higher than about 8 ft and the relief well system is not in operation. River heads higher than 8 ft will increase substratum hydrostatic pressures very little because the top stratum in the bottom of the drainage ditch immediately landward of the levee can withstand only a head of this amount before it ruptures or sand boils develop. Since the project flood stage will create an H of approximately 28 ft, numerous and probably severe sand boils may be expected unless the existing relief well system is operated. Such boils may also be expected downstream of the well system where top stratum conditions are quite similar (see plate 87). No analysis of the condition upstream of the relief well system has been made.

316. The limited h_o that can develop beneath a given top stratum is well illustrated by the plot of M-4-W piezometer readings vs river stage on plate 101. This plot shows how the substratum pressure increases

with increasing river stage up to a certain amount equal to about the critical pressure for the weakest point in the top stratum at which sand boils were observed. In this instance after the river rose above 9 ft on the levee (el 189) the hydrostatic pressure at piezometer M-4-W remained essentially constant.

317. Hydrostatic head vs depth. Several line M and R piezometers were installed at various elevations within the pervious substratum for determining variation in head at depth within the sand stratum, and loss of head as the subsurface seepage rises to the surface. The piezometric head above ground surface as measured by such piezometers at about the crest of the 1951 and 1952 high waters is plotted in fig. 33. The tips

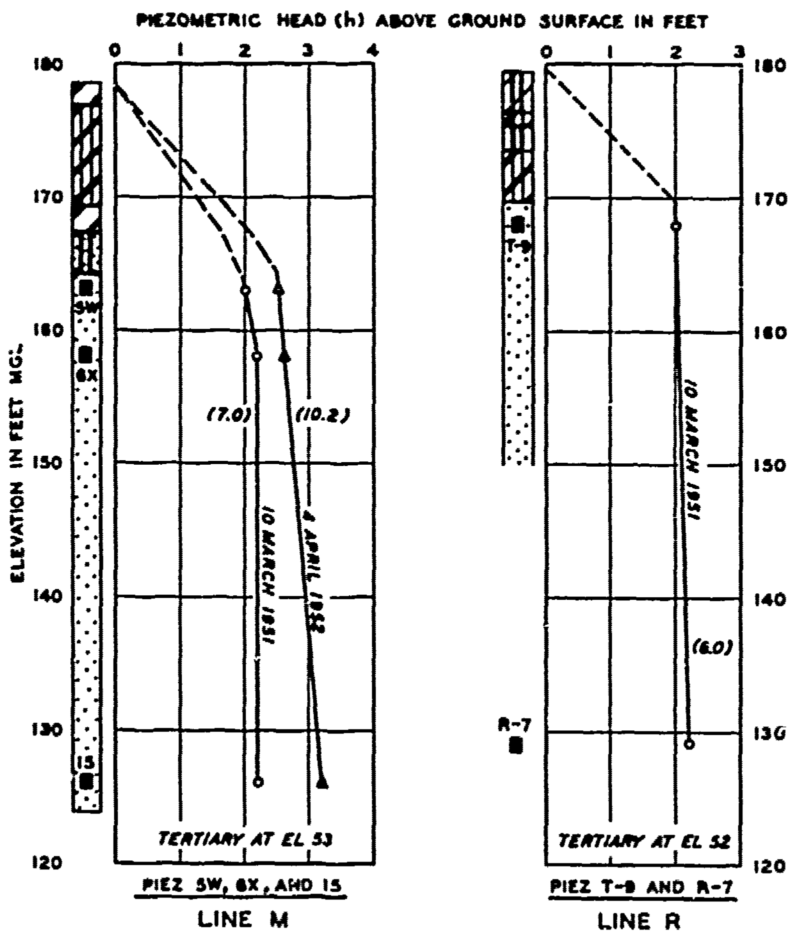

Fig. 33. Piezometric head above ground surface at various depths landward of levee, Trotters 54, Miss. (Note: figures in parentheses are river stage H above ground)

of these piezometers in this case were installed beneath the top stratum and below the fine to very fine sand stratum previously described. Data in fig. 33 indicate relatively little loss in head (0.2 to 0.6 ft) as the seepage from the deep, principal water-carrying stratum rose vertically through the overlying stratum of fine to very fine sand for H = 6 to 10 ft. For high river stages the head loss up through the fine sand stratum would, of course, be proportionately greater.

Evaluation of seepage problem and recommendations for control measures

318. An H of approximately 8 ft at Trotters 54, a river stage approximately 20 ft below project flood stage, causes the formation of sand boils in the drainage ditch landward of the levee. At no time during the high waters since 1943 has any hydrostatic head risen above the surface of the seepage berm at Trotters 54 (see plates 97-99). The predicted gradient beneath the existing levee and seepage berm at line M for the project flood without the relief well system in operation is plotted on plate 98. Thus, the existing seepage berm is not wide enough to prevent the development of critical uplift pressures but has more than adequate thickness.

319. The relief well system on either 50- or 100-ft centers is considered adequate to prevent dangerous sand boils from forming along the reach in which it is installed.

320. In view of the similarity of soil conditions downstream of the relief well system, it is believed that the seepage berm from sta 54/8+25 to 54/28 should be extended landward, or the relief well system extended downstream in order to insure the safety of the levee at project flood stage. No studies have been made regarding conditions upstream of the present relief well system.

Stovall, Mississippi

321. Stovall was originally selected for investigation because it was one of the worst underseepage and sand boil sites in the Memphis District during the 1937 flood. In 1947 it was chosen for installation

of piezometers because of the availability of fairly complete soils data, and the fact that the site represented a highly irregular type of landward top stratum conditions. It was also still considered to be a site that might be critical with respect to underseepage.

Description of site

322. The site is located along the east bank levee of the Mississippi River approximately 3-1/2 miles west of Stovall, Miss., and extends from sta 77/20 to 78/30. The levee is approximately 2500 ft from the bend of Island 63, a chute of the Mississippi River. Plans of the site, river, borrow pits, surface geology, topography, and piezometers are shown on plates 102 and 103. Plate 104 is an aerial mosaic of the site flown in November 1947; an aerial mosaic of the site taken before construction of the present levee and seepage berm is included as an insert on this plate. The location of the landside toe of the levee as it existed during the 1937 high water is shown by a dotted line on the insert. The levee at Stovall has a net height of approximately 29 ft. Extensive riverside borrow pits have been excavated along the portion of levee studied, in some of which the natural top blanket of clays and silts has been removed, while in others there is still an appreciable thickness of clay cover. River stages can be estimated from the Helena, Ark., gage and the graph on plate 116.

323. History of underseepage. During the 1937 high water, a maximum H of 26.5 ft caused heavy underseepage and sand boils between sta 77/25 and 78/30. Five sand boils occurred in the vicinity of the levee toe at sta 77/40; between sta 78/12 and 78/16 the sand boil area extended from the levee toe landward for 200 ft. Opposite sta 78/6 a large sand boil and three smaller boils developed on the bank of a slough 210 ft from the toe of the levee. All of these boils were discharging considerable material when first discovered. The boils were surrounded with large sack sublevees but continued to discharge very fine sand for more than 15 days. The material ejected was deposited to a depth of 6 ft in the slough and amounted to approximately 1000 cu yd. Fig. 9 is a photograph of the large sand boil which is also shown on plate 116. This was probably one of the worst boils that occurred along the Lower Mississippi

River levees during the 1937 high water.

324. A large landside seepage berm about 10 ft thick at the levee toe and 200 ft wide was constructed along the Stovall loop levee in 1938. The location and extent of this berm are shown on plates 102 and 103; typical sections of the berm are shown on plates 105 and 106.

325. During the 1950 high water (maximum H = approximately 15 ft), heavy underseepage occurred along the toe of the seepage berm and landward for about 100 ft between sta 77/25 and 78/30. Ten sand boils varying in size from 2 to 4 in. were active between sta 77/41 and 77/43; their flow was estimated to be from 5 to 70 gpm each. Relatively little sand was discharged from any of the boils. Seepage filled the slough landward of the levee from sta 78/4 to 78/25. Upstream of this slough, most of the seepage and flow from the sand boils were concentrated in a small drainage ditch parallel to and approximately 100 ft from the toe of the berm. Measurement of the flow in this ditch at sta 77/38+75 and 78/1+30 on 21 February 1950 showed that 1400 gpm of seepage was rising to the surface between these two points and the ditch landward of the berm toe. This seepage amounted to 117 gpm per 100 ft of levee with H = 14.5 ft, or 8.1 gpm per 100 ft of levee per ft H .

326. Locations of the sand boils that occurred during the 1937 and 1950 high waters and the location of the seepage measuring points are shown on plate 103.

327. Piezometer installation. In 1948 two lines of piezometers, A and B, were installed perpendicular to the levee at sta 77/38+00 and 77/48+50, respectively, with some piezometers (line E) placed along the berm toe from sta 77/33 to 78/20 (plates 103 and 104). Piezometer readings were obtained during the 1950 high water.

Geology of site and soil conditions

328. The general geology of the site is illustrated on plate 102. The type and thickness of top stratum soils are illustrated in more detail on plate 103. This site is located in an area of point bar and channel deposits formed while the river occupied courses 12 and 13 and as it was gradually enlarging a meander loop (see plate 102). Clayey natural levee deposits, which are indistinguishable in composition from

the underlying fine-grained point bar deposits, cover the area. So far as is known, the natural levee deposits contain no silty or sandy materials.

329. The top stratum consists largely of clayey deposits highly variable in thickness, ranging from about 11 to 30 ft, being thickest in two broad swales that cross the levee at an angle of approximately 45 degrees (plate 103). Although the two broad swales at the site are largely filled with clay, they do contain numerous strata of silts and very fine sands (plate 105). The top stratum between the swales also consists of clay about 10 to 12 ft thick.

330. Relation of underseepage to geology. The most severe underseepage and sand boils occurring during the 1937 high water were located between the then-existing landside toe of the levee and the rather massive clay-filled swales at the site (see plate 103). It is of particular interest to note the location of the very large boils at the edge of the old slough at sta 78/6. It is also of interest that practically no sand boils were observed in the areas of the thick, clay-filled swales. Although the top stratum was 10 to 15 ft thick between the levee toe and the clay-filled swales, the relatively high head on the levee, nearness of source of seepage, and the concentrating effect of the swales landward of the levee caused the formation of very active and serious sand boils in the area during the 1937 flood (see section A-A, plate 105, and C-C, plate 106).

331. The seepage berm constructed in 1938 covered most of the thinner top stratum between the levee toe and the swale between sta 78/5 and 78/15, and no sand boils occurred along this reach of levee during the 1950 high water. However, the surface geology, as shown on plate 103, indicates a potentially critical area between borings S-1 and S-6. Although the seepage berm blanketed the 1937 sand boil area between sta 77/38 and 77/40, numerous active sand boils occurred at the toe of the present seepage berm between the two clay-filled swales during the 1950 high water (see plate 103).

332. It is pointed out that the toe of the present seepage berm is now only about 0 to 500 ft from the edge of the landwardmost

clay-filled swale (see plate 103, section A-A on plate 105, and section C-C on plate 106). While the seepage berm may have eliminated one of the potentially critical sand boil areas, it has not alleviated the situation between sta 77/35 and 77/47, except by lengthening the path of seepage and moving the point of potential piping farther from the levee toe. Another predominating cause of seepage and sand boils in this reach is the close effective seepage entrance in those portions of the riverside borrow pits excavated to sand. The most severe seepage during the 1937 and 1950 high waters was opposite those portions of riverside borrow pits in which most of the clay top stratum had been removed (see plate 103). The fact that no sand boils occurred along other portions of the Stovall site geologically similar to reaches where large sand boils did occur can probably be attributed to the fact that the borrow pits opposite the former reaches of levee are still covered with an appreciable thickness of clay top stratum.

333. Soil profiles and piezometer lines. Locations of piezometers and borings are shown in plan on plates 103 and 104. Soil profiles and piezometer lines are shown on plates 105-107; a soil profile through the riverside borrow pits and logs of miscellaneous borings made in these pits are shown on plate 108. One piezometer line (A) was located perpendicular to the levee at a point considered particularly critical with regard to underseepage. Piezometer line B was located so as to cross one of the rather deep swales at a point where the swale crosses beneath the toe of the present seepage berm and where no serious sand boils had occurred. One piezometer, D-10, was also installed riverward of the levee between lines A and B in a portion of the borrow pit that had been excavated to sand. Other piezometers were installed beneath and along the toe of the present seepage berm at locations considered representative of different conditions at the site. The tips of some of the line B piezometers were installed at an elevation approximately equal to the depth of the swale which this line of piezometers crosses, for the purpose of determining if the pressure head drops significantly across the swale filling (see section B-B, plate 105).

334. The sediments making up the top stratum in the point bar area

at Stovall are quite variable as regards thickness and type. The filling in the large swale farthest landward of the levee consists essentially of an upper stratum of clay approximately 25 ft thick underlain by 5 to 10 ft of sandy silts and silty sands (plate 105). The other swale at the site appears to be filled with more heterogeneous strata of silts, clays, and sands (see section B-B, plate 105, and C-C, plate 106). The top stratum between and adjacent to these swale fillings consists of a rather uniform clay approximately 10 to 12 ft thick underlain by a 2- to 8-ft stratum of silty sands and sandy silts.

335. The pervious substratum at the site consists of medium to coarse sands approximately 50 ft in thickness (plates 106-107). Although this pervious substratum is the thinnest of all sites investigated along the Mississippi River, it has sufficient seepage and pressure-carrying capacity when combined with unfavorable borrow pit conditions and geological features to cause serious sand boils and piping.

Analysis of piezometric and seepage data

336. River stage and piezometer readings observed at Stovall during the 1950 high water are plotted on plates 109 and 110. This high water created a maximum H of about 15 ft. Piezometric gradients existing in the pervious substratum beneath the levee and berm along piezometer lines A and B at selected river stages during the 1950 high water are shown on plates 111 and 112. The hydrostatic head along the toe of the seepage berm (line E) is plotted on plate 113. The excess head along the berm toe amounted to 6 to 9 ft at the crest of the 1950 high water. High excess heads also existed for considerable distances landward of the levee; 1000 ft landward of the berm toe the excess head amounted to as much as 5 ft, or 33% H (see plates 111 and 112).

337. Information pertaining to the site and results of analyses of piezometric and seepage data subsequently discussed are summarized in table 14.

338. Source of seepage. Seepage may enter the pervious substratum at Stovall in a chute of the Mississippi River and in parts of riverside borrow pits where most of the natural clay blanket has been removed (see

Table 14

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Stovall, Miss., Site

Factor	Line A 1950 Flood	Line A Project Flood	Line B 1950 Flood	Line B Project Flood	Design Values Sta 77/26-77/36	Design Values Sta 77/36-77/44	Design Values Sta 77/44-78/5	Design Values Sta 78/5-78/13	Design Values Sta 78/13-78/21
River stage (crest)	179.4*	194.3	179.4	194.3	194.3	194.3	194.3	194.3	194.3
Average el of ground or tailwater	164.5	164.5	164.5	164.5	165.5	164.5	164.5	163.0	163.0
Head on levee (H)	14.9	29.8	14.9	29.8	28.8	29.8	29.8	31.3	31.3
Piezometers used in analysis	B-3 & -4	-----	E-12 & -13	-----	-----	-----	-----	-----	-----
Riverside borrow pit, width, ft	1000	-----	1000	-----	1000	1000	1000	1000	1000
Top stratum	0-3 ft clay**	-----	0-6 ft clay	-----	-----	-----	-----	-----	-----
Average stratum	Sand	-----	Sand	-----	5 ft clay	Sand	Sand	4 ft clay	4 ft clay
Distance from riverside levee toe to river (L_1)	3000	-----	3000	-----	-----	-----	-----	-----	-----
Base width of levee (L_2)	600	-----	600	-----	600	600	600	600	600
Landward extent of top stratum (L_3)	450	-----	900	-----	400	400	900	1300	1400
Distance to effective seepage source (s)	1080	1050	800	800	1200	1000	800	1000	1000
Effective length of riverside blanket (x_1)	480	450	200	200	600	400	200	400	400
Distance to effective seepage exit (x_3)	1360	1150	800	750	1500	1100	850	950	1000
Effective thickness of sand substratum (d)	40	-----	40	-----	40	40	40	40	40
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	2500	-----	2500	-----	2500	2500	2500	2500	2500
Laboratory permeability tests	350	-----	350	-----	-----	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	950	-----	950	-----	-----	-----	-----	-----	-----
Seepage and piezometric data	3370	-----	2320	-----	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	-----	-----	-----	-----	-----	-----	-----
Top stratum, type	Clay	-----	Silty clay	-----	Clay	Clay	Clay	Clay	Clay
Effective thickness for seepage analysis (z_{bL})	12.0	-----	15.0	-----	14.5	12.0	15.0	10.5	18.0
Critical thickness (z_t)	10.5	-----	15.0	-----	12.0	10.5	12.5	10.5	18.0
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	3.1	3.6	3.6	4.2	-----	-----	-----	-----	-----
Piezometric data and blanket formulas	2.2	2.7	3.2	3.6	-----	-----	-----	-----	-----
Piezometric data and seepage measurements	5.3	-----	5.5	-----	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	800	700	700	600	800	800	800	800	1000
Blanket formula	1150	940	780	700	-----	-----	-----	-----	-----
Natural seepage measurements	520	-----	500	-----	-----	-----	-----	-----	-----
Natural seepage beneath levee									
Q_s, gpm/100 ft of levee	90	200	135	280	-----	-----	-----	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	6.1	6.7	9.1	9.4	-----	-----	-----	-----	-----
Q_A/H, gpm/100 ft levee between sta 77/38+75 and 78/1+30 and between levee and L.S. drainage ditch (measured)	-----	-----	8.1	-----	-----	-----	-----	-----	-----

* Maximum river stage was recorded on 21 February 1950 when natural seepage was measured. This river stage is not shown on plates 111-112.

** Riverside swale not included.

plates 102, 103, and 105). No significant seepage was reported during either the 1937 or 1950 high waters along those reaches of levee where the riverward clay blanket had not been removed by borrow pit operations.

339. The distance to the effective source of seepage as determined at piezometer lines A and B during 1950 is plotted in fig. 34. The source of seepage as determined graphically at lines A and B during the crest of the flood is shown on plates 111 and 112, respectively. These plates and fig. 34 show that seepage enters the sand substratum primarily through the riverside borrow pits where the foundation sand has been exposed by borrow operations. The effective distances to the source of seepage at lines A and B as measured from the berm toe were about 1100 and 800 ft, respectively, at the crest of the 1950 flood, or only 300 ft riverward of the riverside toe of the levee at piezometer line B. It appears from fig. 34 that s remained essentially constant as the river rose. The source of seepage is farther from the levee at line A than at line B because of the clay-filled swale at the riverside levee toe at line A (plate 105). It is estimated that s will be about 1050 and 800 ft from the berm toe at lines A and B, respectively, at the project flood.

340. <u>Seepage exit.</u> Values of x_3 determined from piezometer readings are plotted vs corresponding river stages in fig. 34. At line A,

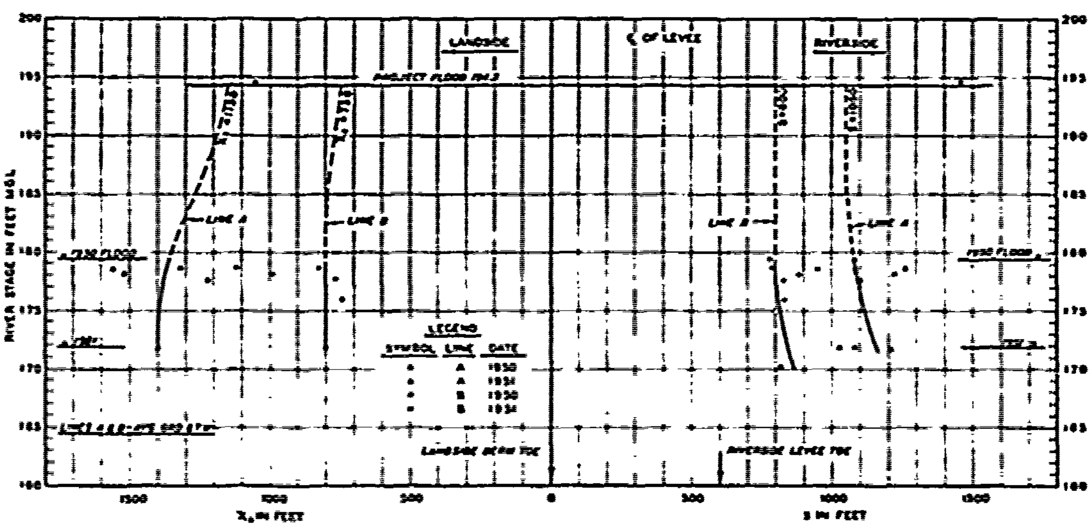

Fig. 34. Distances to effective seepage source and exit. Stovall, lines A and B

where the clay top stratum is about 12 ft thick for about 400 ft landward and then becomes about 25 ft thick, x_3 was about 1350 ft at the crest of the 1950 high water. At line B, where the top stratum consists of about 15 ft of clay and silt, x_3 was about 800 ft during the 1950 flood. It is estimated that x_3 will be about 1150 and 750 ft at lines A and B, respectively, at project flood stage. The greater value of x_3 at line A is attributed to the clay plug landward of the berm toe which in effect blocks the emergence of seepage along this line.

341. Thickness and permeability of substratum sands. The pervious foundation at Stovall consists of an upper stratum of fine sand about 20 to 25 ft thick underlain by about 35 ft of medium-coarse sand (plates 106 and 107). Grain-size curves for typical samples of the foundation sands are shown on plate 116. On the basis of the relationship between D_{10} and k_f (fig. 17) the upper sands have a permeability of about 200 x 10^{-4} cm per sec and the lower sand stratum about 950 x 10^{-4} cm per sec. On the basis of D_{10} the aquifer is considered to have an effective thickness of 40 ft. Based on laboratory permeability tests, k_f would be only 350 x 10^{-4} cm per sec, which is quite low for medium and coarse sands. At the crest of the 1950 high water, natural seepage between the levee and drainage ditch landward of the levee amounted to 117 gpm per 100 ft of levee between sta 77/38+75 and 78/1+30 with $H = 14.5$ ft. From these seepage measurements the permeability of the foundation was estimated to be 3370 x 10^{-4} cm per sec at line A and 2320 x 10^{-4} cm per sec at line B. Weighing k_f obtained from D_{10} data and fig. 17 with that from seepage measurements, the average k_f at Stovall was taken to be 2500 x 10^{-4} cm per sec for the aquifer on the basis of its being 40 ft thick.

342. It should be noted that a discrepancy exists between values of k_f as determined from D_{10} and Q_A, and the differences cannot readily be explained. It is possible that thin gravel strata which were not sampled with either the split spoon or bailer may be present in the pervious substratum. Such gravel strata would have a large seepage-carrying capacity and could result in a highly pervious substratum.

343. Thickness and permeability of top stratum. The average top

stratum at line A consists of 12 ft of clay between the levee toe and a point 450 ft landward of the berm toe, beyond which the thickness increases considerably (plate 105). Beneath the landside ditch 100 ft landward of the berm toe, the top stratum is only about 10.5 ft thick. The clay top stratum at line A is underlain by about 3 ft of silty sand for the first 100 ft landward from the berm toe which increases considerably farther landward of the levee. On the basis that $z_{bL} = 12$ ft, k_{bL} was computed to be 2.2×10^{-4} cm per sec from formula 5, using $x_3 = 1360$ as obtained from the 1950 high-water data and a finite length of landside blanket $L_3 = 450$ ft. From natural seepage measurements the permeability of the top stratum at line A was estimated to be 5.3×10^{-4} cm per sec. Thus, the average k_{bL} at line A during the 1950 high water was about 3.1×10^{-4} cm per sec. At line B the top stratum landward of the berm toe consists of about 15 ft of clay and silt. On the basis that $z_{bL} =$ 15 ft, k_{bL} was found to be 3.2×10^{-4} cm per sec from formula 5, using $x_3 = 800$ ft and $L_3 = 900$ ft. From natural seepage measurements k_{bL} was estimated to be 5.5×10^{-4} cm per sec. The average k_{bL} at line B was about 3.6×10^{-4} cm per sec.

344. Permeability ratio. The ratio of the permeability of the foundation to that of the clay top stratum is estimated to have been 800 at line A, and 700 at line B during 1950. Estimates of k_f/k_{bL} for project flood stage are given in table 14.

345. See_ge flow. Seepage passing beneath the levee at lines A and B at the crest of the 1950 flood, and for the project flood, was estimated using corresponding measured values of H, s, and x_3. Seepage at the 1950 crest was estimated to be about 100 gpm per 100 ft of levee at line A and 150 gpm per 100 ft of levee at line B with an H of about 15 ft. Q_s/H ranged from about 6 to 9 gpm per 100 ft of levee in 1950. The observed seepage Q_A landward of the levee was 117 gpm per 100 ft of levee in an area which encompassed both piezometer lines A and B; $Q_A/H = 8$ gpm. Estimated natural seepage at project flood stage is about 200 gpm at line A and 300 gpm at line B per 100 ft of levee (see table 14). Because of the relatively flat piezometric grade line landward of the existing drainage ditch it is believed that little seepage passes

beyond this point, and that most of it emerges between the berm toe and the ditch. Therefore the seepage measured at Stovall represents most of the seepage that passed beneath the levee during the 1950 flood.

346. Landside substratum pressures. Hydrostatic pressures that developed along the toe of the seepage berm during the crest of the 1950 high water are shown on plate 113. Readings of selected piezometers at or near the landside toe of the berm are plotted vs corresponding river stages on plates 114 and 115, which also show estimated substratum pressures for project flood stage. The head on the levee, type and thickness of top stratum, and substratum pressures at each piezometer along the landside toe of the berm are given in table 15. From plates 109 and 110 it may be noted that changes in river stage were rapidly reflected in readings of piezometers landward of the levee. From plates 114 and 115 and table 15, it appears that uplift pressures sufficient to cause active sand boils will develop along the toe of the present seepage berm between sta 77/20 and 78/8 at flood stages higher than about 15 to 20 ft on the levee. Downstream from sta 78/8, critical uplift pressures probably will not develop until H is higher than about 22 ft, as the top stratum is generally thicker than that upstream (plates 103 and 107). Sand boils

Table 15

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Stovall, Miss., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c ($0.85\ z_t$) ft	Crest of 1950 Flood (178.7)[e]: H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (194.3): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
E	A-1[f]	165.5	-----	14.5	12.5	27.0	19.0	16.2	13.2	8.5	64	----	0.45	----	----	28.8	16.2[c]	56	21.3
		163.0[g]	163.2	11.0[h]	12.0[h]	23.0[h]	15.0[h]	12.8	15.5	10.8	70	0.72	----	----	----	31.1	12.8[c]	41	17.9
A-E	B-5[i]	165.0	-----	11.2	3.5	14.7	11.5	9.8	13.7	6.0	44	----	0.52	----	----	29.3	9.8[c]	33	18.7
A-E	B-5-A	162.7	163.2	10.0[h]	2.5[h]	12.5[h]	10.5	8.9	15.5	---	--	----	----	----	----	31.1	8.9[c]	29	18.7
E	C-9	165.0	-----	15.0	5.0	20.0	15.0	12.8	13.7	6.0	44	----	0.40	----	----	29.3	12.6[c]	44	27.0
		162.5[g]	163.2	12.5[h]	5.0[h]	17.5[h]	12.5	10.6	15.5	7.8	50	0.63	----	----	----	31.1	10.6[c]	34	21.4
B-E	E-14[f]	165.0	-----	3.0	7.0	10.0	10.0	8.5	13.7	1.7	12	----	----	----	0.17	29.3	8.5[c]	29	----
B-E	E-15[f]	165.0	-----	----	---	19.0[j]	19.0	16.1	13.7	7.4	54	----	0.39	----	----	29.3	16.1[c]	55	23.8
B	Avg E-15 E-16	162.2	163.2	----	---	17.5[j]	17.5	14.9	15.5	7.7	50	----	0.44	----	----	31.1	14.9[c]	48	22.1
B	E-16	164.0	-----	14.0	4.0	18.0	14.0	11.9	14.7	6.3	36	----	----	----	0.38	30.3	11.9[c]	39	----
C	Avg G-19 F-18	160.0	163.0	10.5	7.5	18.0	11.0	9.3	15.7	9.8	62	----	0.89	----	----	31.3	9.3[c]	30	----

[c] See paragraph 143.
[e] Maximum river stage in 1950 at which piezometric data were obtained.
[f] In swale.
[g] Invert elevation of drainage ditch.
[h] Top stratum beneath ditch.
[i] At edge of swale near ditch.
[j] Stratified clay and silt.

and medium to heavy underseepage were observed between sta 77/25 and 78/30 in 1950 with an H of about 15 ft. This stage created excess heads along the toe of the levee of 6 to 9 ft, or 40 to 65% H. In areas of sand boils, i ranged from about 0.62 to 0.73. As the project flood stage will create an H of about 30 ft, considerably heavier seepage and more numerous and active sand boils can be expected at the Stovall site than occurred in 1950.

347. The irregular top stratum and shallow landside ditch affect the seepage pattern upstream of sta 78/8. The boils that developed in 1937 and 1950 generally occurred in the thin, intervening ridges between the thick swales and sloughs in this reach. Downstream from sta 78/8 the area for 800 ft landward of the levee appears to have a relatively thick top stratum (a clay-filled slough), and a considerable amount of clay blanket remains in the riverside borrow pits. No sand boils were observed downstream of sta 78/8 during the 1950 high water.

Evaluation of seepage problem and recommendations for control measures

348. An H of 12 to 15 ft caused the formation of rather high uplift pressures and sand boils along the berm toe at Stovall during the 1950 high water. The seepage berm constructed in 1938 has forced the location of sand boils landward from the levee toe but has not eliminated the possibility of sand boils occurring which would result in a potentially critical situation. At line A and between sta 78/0 and 78/10 the berm possibly has potentially increased the severity of the underseepage problem, because it has reduced the area, between the levee and the landward clay-filled sloughs and swales, in which seepage can emerge. Since H will be about 30 ft at the crest of the project flood, it is believed that a critical underseepage condition still exists. An important factor in this evaluation is that the top stratum is highly irregular and composed of ridges and swales inclined with respect to the levee in such a manner that concentrations of seepage can be expected near the edges of the swales. The proximity of the drainage ditch between sta 77/26 and 78/0 also aggravates the seepage condition. Because of the close source of seepage in the sandy riverside borrow pits and the high seepage-carrying

capacity of the pervious foundation, considerable seepage and numerous sand boils can be expected between sta 77/20 and 78/8 when H is higher than 12 ft. In view of this, it is considered necessary that additional seepage control measures be provided at the Stovall site. Such measures could consist of seepage berms or relief wells. Relief wells probably would be more desirable because of the highly irregular nature of the landside top stratum, and because wells can be located at critical points in the top stratum, whereas a berm extending an additional 150 to 200 ft landward would raise the hydrostatic pressures beneath the levee and berm.

349. The existing seepage berm has approximately adequate thickness for its width; however, if the berm is widened, its thickness should be increased somewhat.

Farrell, Mississippi

350. Farrell was selected as one of the original sites for investigation because of its record during the 1937 high water. It was selected subsequently for installation of piezometers because of data available from previous investigations at the site, and because it afforded an opportunity for studying the effect of riverside borrow pits on seepage and substratum pressures, as the river is a considerable distance from the levee at this site.

Description of site

351. The site is located along the east bank levee of the Mississippi River approximately 3 miles west of Farrell, Miss. The reach of levee studied is that between sta 81/10 and 82/20 where the levee is approximately one mile from the bend of Island 63, a chute of the Mississippi River.

352. Plans of site, river, borrow pits, surface geology, topography, and piezometers are shown on plates 117 and 118; plate 119 is an aerial mosaic of the site. Riverside borrow pits 5 to 10 ft deep and 800 to 1000 ft wide exist along the site. The natural top stratum of clay and silt has been materially decreased in these pits by borrow operations and the pervious foundation sands have been exposed at some locations. The

levee at Farrell has a net height of approximately 24.5 ft. River stages can be estimated from the Helena, Ark., gage on the Mississippi River and the graph on plate 116.

353. History of underseepage. Heavy underseepage and sand boils occurred between sta 81/2 and 83/2 during the 1937 high water (maximum H = 23 ft). The levee banquette also settled between sta 81/41 and 81/43. At least 12 sand boils, which discharged considerable sand, were noted within 100 ft of the portion of banquette that settled; locations of these sand boils are shown on plate 118. In 1938 a large landside berm about 10 ft thick at the levee toe and 200 ft wide was constructed. The location and extent of the berm are shown on plates 117 and 118; typical sections are shown on plate 120.

354. During the 1950 high water (maximum H = 7 ft), only light seepage occurred between sta 81/10 and 81/42; however, several pin boils occurred in a drainage ditch approximately 100 ft landward of the berm between sta 81/16 and 81/17. Seepage between the berm toe and the drainage ditch was estimated at about 5 to 10 gpm per 100-ft levee station from sta 81/10 to 81/30. Seepage water covered the low-lying ground from sta 81/30 to 82/10 to a depth of 6 to 12 in.; however, no sand boils were observed along this reach. It is to be noted that the head on the levee during the 1950 high water was very low compared to that during the 1937 high water.

355. Piezometer installation. In 1948 a line of piezometers was installed approximately perpendicular to the levee at sta 81/24 (line A) and several piezometers were placed along the toe of the present seepage berm from sta 81/15 to 82/5 (line C). The tips of the piezometers in line A were set immediately below the top stratum and also below a thin seam of clay underlying the area immediately landward of the levee at a depth of about 25 ft (plate 120). Two line A piezometers were installed riverward of the levee in the riverside borrow pits. Piezometer readings were obtained during the 1950 high water.

Geology of site and soil conditions

356. The general surface geology is depicted on plates 117 and 118. Plate 117 shows the location of former river courses, swales, and

natural levee deposits blanketing the area. A more detailed picture of the character and thickness of the top stratum, and the location of underseepage with respect to geological features are shown on plate 118.

357. The site is located mainly on point bar deposits laid down as the river gradually enlarged a meander loop during river courses 13 to 15. A broad chute which linked the two arms of the meander loop developed during course 15. Later, course 15 was cut off from the main channel of the river. The old channel and chute subsequently were filled with silts and clays, which encircle the Farrell site on three sides. An exceptionally broad swale, which branches into four smaller units, crosses the levee at an angle of approximately 90 degrees. Three rather large swales, 25 to 30 ft deep and filled with alternating strata of clays, silts, and sands, cross the levee between sta 81/28 and 82/1. The top stratum immediately landward of the levee is extremely variable in thickness and types of material (plates 118 and 121). The top stratum from sta 81/10 to 81/28 consists of clays and silts 2 to 5 ft thick. The top stratum between the two upstream swales consists of clays and clay silts approximately 8 to 15 ft thick; between the two downstream swales the top stratum consists of silty sands approximately 7 to 10 ft thick. Downstream of sta 81/1 the top stratum is composed of an upper, very thin stratum of clay 1 to 3 ft thick underlain by sand and silty sand to a depth of approximately 14 ft. Natural levee deposits of the same general composition as the underlying fine-grained top stratum appear to cover much of the area.

358. Relation of underseepage to geology. Although the river is approximately one mile from the levee, heavy underseepage and sand boils occurred along this reach of levee, owing to the deep riverside borrow pits, the very thin top stratum in many places, and the high head existing during the 1937 high water. The worst sand boils occurred in a small area between two broad, clay-filled swales where the top stratum was thin and seepage from the deep underlying pervious substratum was forced to concentrate. It was in this area that there was apparently sufficient piping to cause the levee banquette to settle. The effect of the deep clay-and-silt-filled chute bordering the site on the southeast is difficult

to assess, but this chute may aggravate the problem by preventing the emergence of subsurface seepage. Although the seepage berm constructed in 1938 has covered the location of most of the 1937 sand boils, the same basic conditions remain that tend to cause underseepage and sand boils, except for whatever reduction in severity is obtained from an increase of 200 ft in seepage path length. Of course, the seepage berm has moved the point of potential boils farther landward of the levee toe. The influence of discontinuities in thickness and composition of the top stratum on localization of sand boils and underseepage is well illustrated at this site.

359. Soil profiles and piezometer lines. Soil profiles at piezometer lines A and C are shown on plates 120 and 121, respectively. Soil profiles along sections B, D, and E are shown on plates 120-122. Logs of miscellaneous borings made in the borrow pits are included on plate 120. A deep boring, F-1, to Tertiary at the toe of the present seepage berm at sta 81/41 shows a pervious sand stratum with a thickness of about 70 ft. Other deep borings in the area show that this sand stratum underlies the entire area. This sand stratum is typical of the pervious alluvium, and consists of alternating strata of sands grading from fine to coarse with some gravel at depth.

360. Piezometer line A was located perpendicular to the levee about 600 ft upstream of the point where one of the broad clay-filled swales crosses under the levee and in an area where the landside top stratum is rather thin. The riverside top stratum has been removed down to the sand foundation as a result of borrow operations. As may be seen from plates 118 and 120, piezometer line A extends from the riverside edge of the borrow pit to a point approximately 1500 ft landward of the levee center line. During installation of the piezometers along this line, a thin seam of clay 1 to 2 ft thick was encountered at approximately 25-ft depth. In view of this unanticipated stratum it was decided to place the tips of most of the piezometers immediately below it, as shown on plate 120. However, the tips of piezometer B-6 and those along the berm toe were placed immediately beneath the top stratum, as shown on plate 121. Subsequent to the 1950 high water another piezometer, B-6-A,

was installed on line A immediately beneath the thin clay top stratum about 100 ft landward of the seepage berm toe (plate 120). Before another major high water occurs, shallow piezometers should be installed along the toe of the seepage berm at the following critical points: sta 81/29, 81/40, and 82/12+50

Analysis of piezometric and seepage data

361. River stages and piezometer readings observed at the Farrell site during the 1950 high water are plotted on plate 123. At the crest of this high water the maximum H was as high as 11 ft and averaged about 7 ft for the site. Piezometric gradients in the pervious substratum beneath the levee at piezometer lines A and C are shown on plates 124 and 125, respectively. Excess heads of 2 to 7 ft developed at the toe of the landside berm between sta 81/20 and 82/3. The shallow piezometer beneath the thin surface top stratum at line A indicated only about 1 ft of excess head. The head beneath the thin clay seam at about el 142 was approximately 2.5 ft higher than that beneath the upper top stratum at the berm toe. The hydrostatic grade line at line A was relatively flat landward of the levee, and excess pressures beneath the deep clay seam were recorded as far as 1000 ft landward of the berm toe.

362. The analyses of piezometric and seepage data reported herein pertain primarily to conditions at piezometer line A which, because of the presence of a deep clay seam, may not be typical of conditions at the Farrell site. As only one piezometer line was installed perpendicular to the levee at this site, values of s , x_3 , k_{bL} , and Q_s downstream from sta 81/30 could not be determined. However, values required to design seepage control measures downstream from sta 81/30 have been estimated and are given in table 16.

363. A summary of information pertaining to the site and results of analyses of piezometric and seepage data are given in table 16. As a result of the apparent continuity of the clay seam at el 142 in a direction perpendicular to the levee, it was necessary to evaluate seepage flow in the sand strata above and below the clay seam. In table 16, the sand above the clay seam is classified as the "upper aquifer," and

Table 16

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Farrell, Miss., Site

Factor	Line A				Design Values				
	Lower Aquifer		Upper Aquifer		Sta 81/10-81/30				
	1950 Flood	Project Flood	1950 Flood	Project Flood	Lower Aquifer	Upper Aquifer	Sta 81/30 to 81/42	Sta 81/42 to 81/48	Sta 81/48 to 82/13
River stage (crest)	174.8	192.1	174.8	192.1	192.1	192.1	192.1	192.1	192.1
Average el of ground or tailwater	168.0	168.0	168.0	168.0	166.6	165.6	164.0	164.0	166.0
Head on levee (H)	6.8	24.1	6.8	24.1	25.5	26.5	28.1	28.1	26.1
Piezometers used in analysis	B-4, -5, -7	-----	-----	-----	-----	-----	-----	-----	-----
Riverside borrow pit, width, ft	850	-----	-----	-----	850	850	850	-----	-----
Top stratum	0-5 ft Si Sd	-----	-----	-----	-----	-----	-----	-----	-----
Average stratum	2 ft Si Sd	-----	-----	-----	2 ft Si Sd	2 ft Si Sd	6 ft Clay	-----	-----
Distance from riverside levee toe to river (L_1)	5000	-----	-----	-----	5000	5000	5000	-----	-----
Base width of levee (L_2)	500	-----	-----	-----	500	500	500	-----	-----
Landward extent of top stratum (L_3)	∞	-----	-----	-----	∞	∞	∞	-----	-----
Distance to effective seepage source (s)	660	800	580	550	800	575	1000	1000	900
Effective length of riverside blanket (x_1)	360	300	80	50	300	-----	500	500	400
Distance to effective seepage exit (x_3)	1000	1050	140	120	1000	135	1100	1500	1000
Effective thickness of sand substratum (d)	70	-----	20	-----	70	20	70	70	70
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1000	-----	300	-----	1000	300	1000	1000	800
Laboratory permeability tests	800	-----	60	-----	-----	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	1200	-----	350	-----	-----	-----	-----	-----	-----
Seepage and piezometric data	-----	-----	-----	-----	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	-----	-----	-----	-----	-----	-----	-----
Top stratum, type	20 ft of sand between thin seams of clay					Clay	Clay	Clay	Silt
Effective thickness for seepage analysis (z_{bL})	6*	-----	4	-----	6	4	7	16	10
Critical thickness (z_t)	25**	-----	3	-----	25	3	7	16	10
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	0.49	0.38	1.22	1.67	-----	-----	-----	-----	-----
Piezometric data and blanket formulas	0.49	0.38	1.22	1.67	-----	-----	-----	-----	-----
Piezometric data and seepage measurements	-----	-----	-----	-----	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	2050	2630	245	180	2400	225	2500	2000	1500
Blanket formula	2050	2630	245	180	-----	-----	-----	-----	-----
Natural seepage measurements	-----	-----	-----	-----	-----	-----	-----	-----	-----
Natural seepage beneath levee									
Q_s, gpm/100 ft of levee	37.5	134	8.2	32	-----	-----	-----	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	5.5	5.6	1.2	1.3	-----	-----	-----	-----	-----

* z_{bL} taken as total thickness of clay strata.
** z_t includes thickness of sand above el 142.

the sand stratum below the clay seam as the "lower aquifer."

364. Source of seepage. Values of s were determined at piezometer line A for both the upper and lower aquifers, and are plotted in fig. 35. These values were necessarily based in part on piezometer B-4, which has its tip at about the elevation of the clay seam. As the head recorded at piezometer B-6 was less than that at B-7, the values of s obtained for the upper aquifer are less than those for the lower aquifer. Had a piezometer been installed immediately beneath the top stratum at the same location as piezometer B-4, it probably would have indicated a lower head than that recorded by B-4, and s might actually be the same for both aquifers. In view of this uncertainty, values of s are given for both aquifers, although those given for the lower aquifer probably are more reliable. At the crest of the 1950 high water, s was 580 and

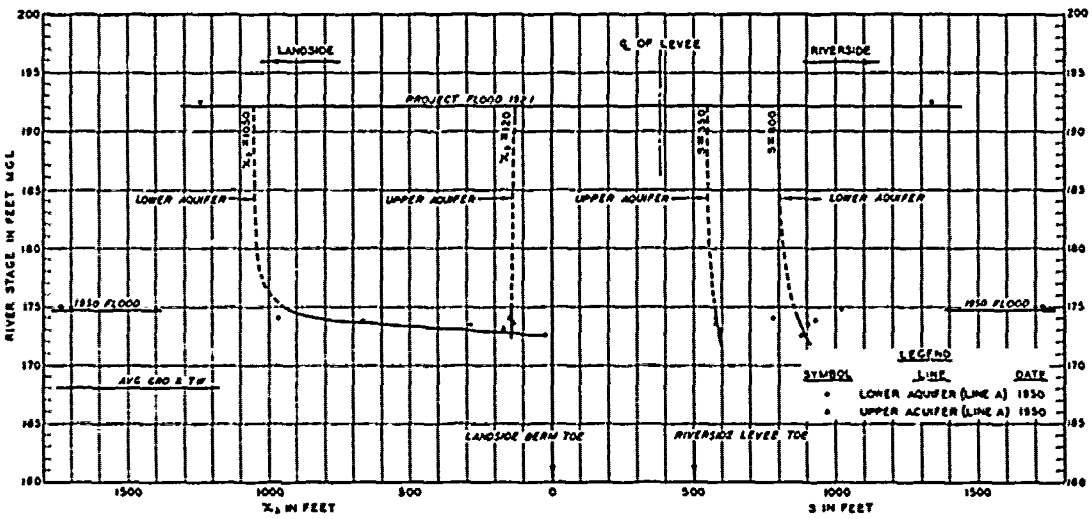

Fig. 35. Distances to effective seepage source and exit. Farrell, line A

860 ft for the upper and lower sand aquifers, respectively. It is estimated that s will be about 550 ft for the upper aquifer at project flood stage and 800 ft for the lower aquifer. On the basis of these values of s it appears that seepage enters the sandy substratum primarily through the borrow pits where most of the natural top stratum has been removed. Relatively little of the seepage passing beneath the Farrell levee is considered to originate in the channel of bend of Island 63, approximately one mile from the site.

365. Seepage exit. Values of x_3 for line A are plotted vs corresponding river stages in fig. 35 for the 1950 high water for the upper and lower sand aquifers. The seepage exit for the upper aquifer was only 140 ft from the berm toe. This close exit is attributed to the very thin top stratum near the berm toe and beneath the landside drainage ditch. It should be noted that x_3 may actually be slightly greater than indicated, because piezometer B-4 was used to evaluate x_3 in the absence of a shallow piezometer (tip immediately beneath the top stratum) beneath the levee. For the lower aquifer, x_3 increased rapidly with rising river stages to a value of about 930 ft at the crest of the 1950 flood. This increase in x_3 is attributed to increased resistance to the flow of seepage landward for filling ground-water storage as the river rose

above bankfull stage. The larger value of x_3 obtained for the lower aquifer is attributed to the lower clay stratum along line A. It is estimated that x_3 will be about 120 ft for the upper aquifer at the project flood and about 1050 ft for the lower aquifer. Little change in x_3 is anticipated for the lower aquifer after the foundation becomes completely saturated.

366. Thickness and permeability of substratum sands. The pervious foundation at Farrell generally consists of alternating strata of medium to coarse sand with strata of finer sands in the upper portion of the sand aquifer (plates 120-122). At piezometer line A (sta 81/24) substratum sands above the clay seam at el 142 are medium-fine; below the clay seam, medium and coarse sands are present (plate 121). The lower aquifer has an effective thickness of about 70 ft; at line A the upper aquifer has an effective thickness of about 20 ft. Grain-size curves for typical samples of both aquifers are shown on plate 116. The permeability of the substratum was estimated from laboratory permeability tests on remolded samples of sand obtained with a bailer (plates 120 and 121), and correlation of k_f vs D_{10} (fig. 17). The results of these determinations are summarized in table 16. A value of 1000 x 10^{-4} cm per sec was selected for k_f for the lower aquifer. The permeability of the upper aquifer (300 x 10^{-4} cm per sec) was based primarily on the correlation given in fig. 17, as the laboratory permeability test data appeared to be too low.

367. Thickness and permeability of top stratum. The top stratum along the berm toe at Farrell is highly irregular as a result of the various swales and ridges along the levee. At piezometer line A, z_{bL} was taken as 4 ft for the top stratum above the upper aquifer. For the lower aquifer z_{bL} was taken as 6 ft, the total thickness of clay above the lower aquifer. The thickness of the intervening upper sand stratum was not considered in evaluating z_{bL} for the lower aquifer, as the vertical permeability of the sand was believed to be sufficiently great as compared to that of the two clay strata to have practically no effect on z_{bL}. However, in evaluation of z_t for the lower aquifer, the thickness of the upper sand stratum was included because it acts as a surcharge on the lower clay seam in resisting uplift. For $z_{bL} = 4$ ft,

k_{bL} was found to be 1.2×10^{-4} cm per sec at line A at the crest of the 1950 high water (see "upper aquifer," table 16). For the lower aquifer, where $z_{bL} = 6$ ft, k_{bL} was about 0.5×10^{-4} cm per sec. The lower permeability of the combined clay stratum (k_{bL} for "lower aquifer," table 16) is attributed to the effect of the lower clay stratum, which probably is not as fissured as the surface clay stratum. Based on the above thicknesses and permeabilities the vertical permeability of the lower clay seam itself is 0.23×10^{-4} cm per sec. (The computed values of k_{bL} for the top stratum above the lower sand aquifer are probably very approximate, as the continuity of the lower clay seam is unknown.) Estimated values of k_{bL} for the project flood are given in table 16.

368. Permeability ratio. Values of k_f/k_{bL} at line A are estimated to have been 245 and 2050 for the upper and lower aquifers, respectively, at the 1950 high water crest. Estimates of k_f and k_{bL} at the crest of the project flood are given in table 16.

369. Seepage flow. Seepage passing beneath the levee at line A at the crest of the 1950 high water was estimated to be about 8 and 38 gpm per 100 ft of levee in the upper and lower aquifers, respectively. These values correspond to values of Q_s/H of 1.2 and 5.5 gpm per ft H . In the vicinity of piezometer line A, most of the seepage in the upper aquifer was probably emerging landward of the levee, whereas the emergence of seepage in the lower aquifer probably was not detectable, as such seepage was very likely flowing laterally into ground-water storage for the river stages experienced. The respective estimated seepage in the upper and lower aquifers at the project flood is 32 and 134 gpm per 100 ft of levee. From these data it is concluded that seepage is comparatively light in the vicinity of line A at the Farrell site.

370. The continuity of the clay seam found at line A at el 142 along the levee is not known, although it appears to be relatively continuous perpendicular to the levee as shown on plate 124. If the clay seam is not continuous between sta 81/10 and 81/30, heavier seepage will generally occur in this reach as compared to that in the vicinity of line A. Downstream from sta 81/30, Q_s may also be greater than that at line A, as no deep clay seam is present.

371. Landside substratum pressures. The hydrostatic pressures which developed along the toe of the berm (line C) near the crest of the 1950 flood are shown on plate 125. Readings of selected piezometers at the berm toe vs river stages are plotted on plate 126; also shown are the estimated substratum pressures for the project flood and the computed maximum piezometer readings based on $i_c = 0.85$. The head on the levee, top stratum characteristics, substratum pressures, and gradients through the top stratum at certain typical piezometers along the landside toe of the berm are given in table 17.

372. From plate 123, it can be seen that two shallow piezometers (A-1 and B-6) lagged considerably behind the river during the initial rise in river stage. Head above the ground surface did not develop at these piezometers until about 18 days after the river reached overbank stages. The data shown on plate 126 and in table 17 indicate that uplift pressures in the upper aquifer sufficient to cause sand boils will develop along the berm toe between sta 81/12 and 81/30 at H higher than 9 to 10 ft. The nonoccurrence of large sand boils in this reach during the 1950 high water is attributed to the low head on the levee (H = 7 ft). The uplift pressures that developed beneath the surface top stratum in 1950 varied from about 1.3 to 2.7 ft between sta 81/12 and 81/30, with the

Table 17

Head on Levee, Top Stratum, Substratum Pressures, and Gradients through Top Stratum along Toe of Levee

Farrell, Miss., Site

Piez Line	Piez Number	Avg Grade at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c (0.85 z_t) ft	Crest of 1950 Flood: H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood: H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
C	A-1	168.2	----	1.3	5.0	6.3	6.3	5.4	6.6	2.0	30	----	----	0.33	----	23.9	5.1	21	9.3
		167.5[e]	----	1.2	5.0	6.2	6.2	5.3	7.6	2.7[f]	35	----	0.45	----	----	25.3	5.1[f,g]	21	10.0
A-C	B-6	167.5	----	3.0	0.0	3.6	3.2	2.3	7.3	1.3	18	----	----	0.43	----	24.6	2.5[f]	10	9.5
		166.2[e]	----	2.8	0.2	2.8	2.5	2.4	8.0	2.4[f]	30	----	0.86	----	----	25.9	2.4[f,g]	9	10.3
A-C	B-7	167.5	----	----	----	25.0[f]	25.0	21.2	7.3	4.7	64	----	----	----	0.19	24.6	17.5[g]	71[f]	36.6
		166.2[e]	----	----	----	23.5[f]	23.5	20.0	8.6	6.2[f]	72	----	----	----	0.25	25.9	----	--	27.9
A-C	B-6-A	167.5	----	3.0	0.0	4.0	3.0	2.5	7.3	---	--	----	----	----	----	24.6	2.5[f]	[illegible]	----
		166.2[e]	----	2.5	0.0	2.5	2.5	2.1	8.6	---	--	----	----	----	----	25.9	2.1[f,g]	8	----
[illegible]	C-11	164.5	[illegible]	----	----	15.0[h]	15.0	12.7	10.3	7.5	73	----	----	0.50	----	27.6	12.7[f]	46	19.2
		163.0[e]	160.0	----	----	13.5[h]	13.5	11.5	10.8	8.0[f]	74	----	----	0.59	----	28.1	11.5[f,g]	41	15.7
C	D-12	163.0	160.0	16.5	5.0	21.5	16.5	14.0	10.8	7.2	67	----	----	----	0.47	28.1	14.0	[illegible]	----
		162.5[e]	160.0	16.0	5.0	21.0	16.0	13.6	10.8	7.2[f]	67	----	----	----	0.45	28.1	13.6[f,g]	48	----
C	E-13	168.0	----	1.0	12.5	13.5[i]	13.5	11.5	6.6	1.2	18	----	----	----	0.09	24.1	11.5	48	16.0
		167.0[j]	----	0.5	12.0	12.5	12.5	10.6	7.6	2.2[j]	29	----	----	----	0.18	25.1	10.6[f,j]	42	15.0

e, f See paragraph 143.

e Bottom of drainage ditch 100 ft landward of piezometer.

f This clay seam overlain by approximately 20 ft of sand and 3 ft of clay.

g Bottom of drainage ditch adjacent to piezometer.

h Stratified clay and silty sand.

i Six feet silty sand overlain by 7 ft fine sand with clay seams.

j Bottom of drainage ditch 75 ft landward of piezometer.

higher heads being those referred to the water in the landside drainage ditch. Upward gradients of about 0.3 to 0.9 occurred in this reach. The excess head beneath the clay seam at el 142 was about 6 ft, corresponding to about 70% H .

373. Downstream from sta 81/30, the estimated river height required to cause sand boils, assuming $i_c = 0.85$, is about 14 ft (compared with H = 26 ft at the project flood). Although H = 10 ft at the crest of the 1950 flood, seepage was comparatively light. Excess heads of 2 to 7 ft were recorded in this reach and correspond to upward gradients of about 0.2 to 0.6. The higher gradients occurred beneath the bottom of the landside drainage ditch.

Evaluation of seepage problem and recommendations for control measures

374. An H of 7 to 11 ft during the 1950 high water did not cause any significant sand boils. Only medium to light seepage was noted. However, from analysis of soil and piezometric data, it appears that uplift pressures sufficient to cause sand boils may occur along some reaches of the levee when H is higher than 10 to 15 ft. H during the 1950 high water was low compared to that in 1937 when serious seepage occurred. The landside seepage berm constructed since 1937 no doubt has improved the safety of the levee and has forced the point of possible subsurface piping farther away from the levee. However, even with the berm in place, active sand boils can probably be expected landward of the berm, particularly between swales, at high river stages.

375. Because of the comparatively thin top stratum upstream between sta 81/10 and 81/30, heavy seepage can be expected along this reach of levee unless the clay seam at el 142 (at piezometer line A) is continuous along the reach. At high river stages, concentration of seepage can be expected along the edges of the swales downstream of sta 81/30.

376. In view of the above and because excavations in riverside borrow pits along a portion of the site have exposed the foundation sands, resulting in a very close source of seepage, additional seepage control measures are considered necessary. Measures recommended are an extension of the berm between sta 81/10 and 81/30 and a line of relief wells along

the toe of the present seepage berm downstream from sta 81/30.

377. The estimated hydraulic gradient at the project flood indicates that in the lower aquifer considerable head may be expected above the berm. This condition is not considered critical if the clay stratum at el 142 is continuous. However, the continuity of this clay stratum is questionable and there probably are reaches of top stratum between sta 81/10 and 81/30 where such a clay seam does not exist. In such reaches the present berm would have sufficient thickness but probably insufficient width for adequate seepage control. Downstream from sta 81/30 the berm appears to be sufficiently thick but may not be sufficiently wide in certain reaches.

378. Before any additional control measures between sta 81/10 and 81/30 are designed, some additional deeper borings should be made along the berm toe to determine the continuity of the clay seam at el 142 along the levee. The existence, or lack thereof, of this seam would have considerable effect on the design of additional control measures.

Upper Francis, Mississippi

379. Upper Francis was selected as a site for study as being representative of a relatively uniform, thick deposit of clay over a broad area where no serious seepage had occurred during the 1937 high water and riverside borrow pits do not penetrate to the underlying pervious substratum. Also this site provided an opportunity to obtain piezometric data beneath an old channel filling where it was thought that the effect of differing geological features at the edges of the channel fillings would not be significant (plate 127).

Description of site

380. The site is located along the east bank levee of the Mississippi River approximately 3 miles west of Mississippi State Highway 1 where the county line between Coahoma and Bolivar Counties crosses the levee. The levee is approximately one to two miles from the Mississippi River. Plans of the site, river, borrow pits, sublevee basin, surface geology, topography, and piezometers are shown on plates 127 and 128;

plate 129 is an aerial mosaic of the site.

381. At the Upper Francis site the levee crosses a former course of the Mississippi which is now filled with a rather thick deposit of clay. Although rather deep and wide borrow pits have been excavated riverward of the levee, they do not penetrate to the underlying pervious foundation. A generalized soil profile perpendicular to the levee at the site is shown on plate 130. The levee has a net height of approximately 25 ft. An approximate relation between river stages at Upper Francis and the Helena, Ark., gage as determined from previous high waters is shown on plate 140.

382. History of underseepage. Although a maximum H of 18.5 to 20.5 ft developed during the 1937 high water, no seepage of any consequence was reported. H was about 11 ft during the 1945 high water; and although seepage was observed for a distance about 500 ft landward of the levee, no boils were noted. In 1948 a large landside seepage berm about 10 ft thick at the toe of the levee and 200 ft wide was constructed. The location and extent of the berm are shown on plates 127 and 128 and typical sections are shown on plates 131 and 132.

383. Some very light seepage was observed in the area landward of the levee during the 1950 high water (plate 128) when H was approximately 8 to 10 ft. The seepage was so light that it could not be measured.

384. Piezometer installation. In 1948 two lines of piezometers, B and C, were installed perpendicular to the levee at sta 39+00 and 57+00, respectively. One piezometer, B-1, was installed in the riverside borrow pits; another, A-1 (line G), was installed at the toe of the seepage berm at sta 30+00. The tips of most of the piezometers were installed immediately below the clay top stratum except for piezometer C-6 which was installed at a depth of approximately 20 ft below the top of the sand substratum. Piezometer readings were obtained during the high water of 1950.

Geology of site and soil conditions

385. The general surface geology is shown on plates 127 and 128. Plate 127 shows the location of former river courses, swales, and natural levee deposits which blanket the area. The character and thickness of

the top stratum in the immediate vicinity of the site are shown in more detail on plate 128.

386. The site is located mainly on thick clays and silts deposited in a cutoff channel of former Mississippi River course 11 (see plates 127 and 131). In consequence, the top stratum is predominantly clayey. The filling in the old channel consists of clays 10 to 20 ft thick underlain by silts, sandy silts, and silty sands 3 to 20 ft thick (plate 134). The area beneath the levee and immediately landward is covered with natural levee deposits of clay silts and sandy silts 2 to 7 ft thick (plates 132 and 134). These natural levee deposits extend from the outside of the bend of course 15 to about 1500 ft landward of the levee.

387. Relation of underseepage to geology. The top stratum at this site is of such character and thickness that no underseepage of consequence has developed for the river stages created by the 1937 and 1950 high waters. The lack of a ready entry for seepage into the pervious substratum on the riverside was no doubt an important factor in preventing underseepage and the formation of sand boils at certain weak points in the top stratum. The only seepage actually observed rising to the surface landward of the levee at this site was at the intersection of piezometer line B and a dirt road approximately 1500 ft landward of the seepage berm where there was an excess head of approximately 5 ft above the ground surface at the crest of the 1950 high water.

388. Some minor seepage probably occurs as a result of percolation through the natural levee deposits of sandy silt that lie immediately beneath the base of the levee.

389. Soil profiles and piezometer lines. Soil profiles at piezometer lines B and C perpendicular to the levee, and line F parallel to the toe of the seepage berm, are shown on plates 131, 132, and 134, respectively. Soil profiles along sections A, E, G, H, and I are shown on plates 132 and 133. Deep borings to Tertiary along the toe of the levee show a pervious sand stratum about 140 ft thick. Other deep borings show this stratum to underlie the entire area. The pervious substratum consists of an upper zone of relatively fine sands approximately 20 ft thick underlain by medium to coarse sands with some gravel; some strata of fine

sands exist throughout the more pervious and coarser sands and gravel. As may be seen from the grain size and permeabilities of the deep sands, the pervious aquifer at Upper Francis has a very high seepage-carrying capacity.

Analysis of piezometric and seepage data

390. River stages and piezometer readings observed at the Upper Francis site during the 1950 high water are plotted on plates 135 and 136. At the crest of this high water, H was about 8 to 10 ft. Piezometric gradients existing in the pervious substratum beneath the levee along piezometer lines B and C at selected river stages are shown on plate 137. The hydrostatic head along the toe of the levee as measured by piezometers along line G is also plotted on plate 137.

391. The excess hydrostatic head along the toe of the seepage berm ranged from about 1.5 to 6 ft at the crest of the 1950 high water. Excess heads as high as 4 ft above the ground surface existed as far as 3000 ft landward of the levee at piezometer line B (see plate 137). These excess heads are based on piezometers installed immediately beneath the clay top stratum at this site.

392. A summary of information pertaining to the site and the results of analysis of piezometric and seepage data subsequently discussed are given in table 18.

393. Source of seepage. The only free entry for seepage into the pervious foundation at the site is in the channel of the Mississippi River some one or two miles distant. The only other source of seepage is through the natural top blanket of silts and silty sands riverward of Parker Bayou (see plate 127).

394. Values of s at piezometer lines B and C during the 1950 high water are plotted in fig. 36. The values of s shown in fig. 36 and on plate 137 indicate that seepage enters the sand substratum primarily through the silty sand top stratum that exists riverward of Parker Bayou (see plates 128 and 131). At piezometer lines B and C s was about 1500 and 1700 ft, respectively, at the crest of the 1950 high water (fig. 36 and table 18). It may be seen from fig. 36 that s changed relatively

Table 18

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Upper Francis, Miss., Site

Factor	Line B: 1950 Flood	Line B: Project Flood	Line C: 1950 Flood	Line C: Project Flood	Design Values: Sta 28-47	Design Values: Sta 47-63
River stage (crest)	168.3	185.0	168.3	185.0	185.0	185.0
Average el of ground or tailwater	160.0	160.0	158.0	158.0	160.0	158.0
Head on levee (H)	8.3	25.0	10.3	27.0	25.0	27.0
Piezometers used in analysis	B-2, B-3, B-4, B-5	-----	C-1, C-3, C-4, C-6	-----	-----	-----
Riverside borrow pit, width, ft	300	-----	300	-----	300	-----
Top stratum	4-10 ft clay 5-8 ft silt	-----	6-10 ft clay 4 ft silt	-----	-----	-----
Average stratum	8 ft clay	-----	8 ft clay	-----	8 ft clay	300
Distance from riverside levee toe to river (L_1)	6800	-----	6500	-----	7000	6500
Base width of levee (L_2)	450	-----	450	-----	450	-----
Landward extent of top stratum (L_3)	-	-----	-	-----	-	-
Distance to effective seepage source (s)	1520	1500	1720	1700	1500	1700
Effective length of riverside blanket (x_1)	1070	1050	1270	1250	1050	1250
Distance to effective seepage exit (x_3)	1400	2000	1540	1500	1950	1450
Effective thickness of sand substratum (d)	125	-----	115	-----	125	115
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1400	-----	1400	-----	1400	1400
Laboratory permeability tests	900	-----	900	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	1900	-----	1900	-----	-----	-----
Seepage and piezometric data	-----	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	-----	-----	-----	-----
Top stratum, type						
Effective thickness for seepage analysis (z_{bL})	18.0	-----	15.0	-----	16	14
Critical thickness (z_t)	18.0	-----	12.0	-----	16	12
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	1.6	0.9	1.1	1.2	0.8	1.0
Piezometric data and blanket formulas	1.6	0.9	1.1	1.2	-----	-----
Piezometric data and seepage measurements	----	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	870	1600	1270	1200	1500	1200
Blanket formula	870	1600	1270	1200	-----	-----
Natural seepage measurements	-----	-----	-----	-----	-----	-----
Natural seepage beneath levee						
Q_s, gpm/100 ft of levee	73	185	75	200	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	8.8	7.4	7.3	7.5	-----	-----

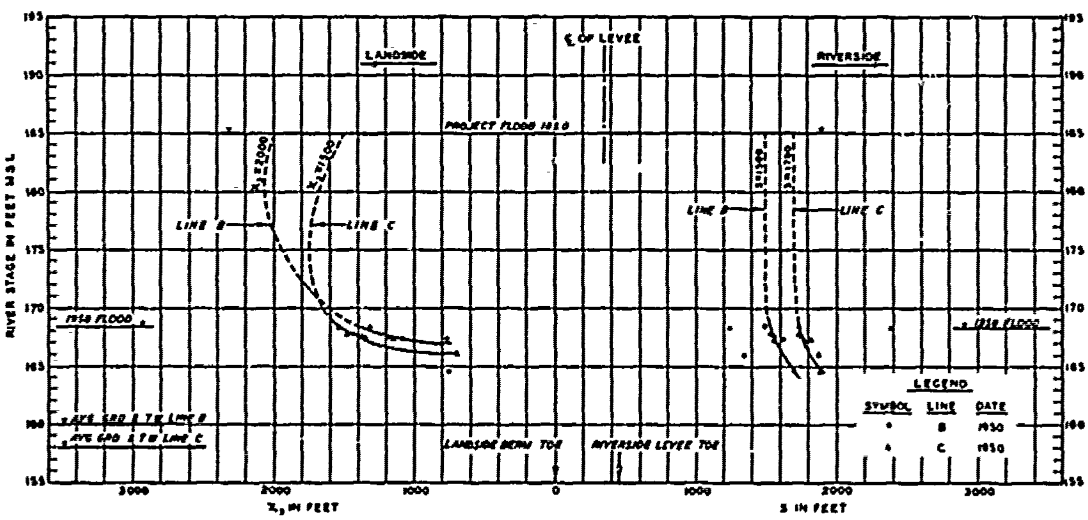

Fig. 36. Distances to effective seepage source and exit. Upper Francis, lines B and C

little as the river rose. Little change in the distance to the effective source of seepage entry with rising river stages would be expected at this site because seepage enters the pervious foundation primarily through the natural top stratum riverward of the levee. The bottoms of borrow pits along the Upper Francis site are blanketed by rather thick clay. For these reasons it was assumed that s would be approximately the same for the project flood stage as the value measured at the crest of the 1950 high water.

395. Seepage exit. Values of x_3 are plotted vs corresponding river stages in fig. 36 for the 1950 high water. The slope of the ground surface away from the berm toe makes a correct determination of x_3 difficult. Assuming average ground and tailwater elevations as shown in the lower left-hand corner of fig. 36, x_3 appears to increase progressively during rising river stages for the heads experienced (8 to 10 ft). The shape of the x_3 curves extrapolated from the observed data to the project flood stage is based on observed data and the maximum computed hydrostatic head that can exist landward of the levee at a project flood stage. The above phenomenon of an increasing x_3 followed by a decrease is similar to that at line E, Trotters 51.

396. The distance to the effective seepage exit at lines B and C at the crest of the 1950 high water was about 1500 ft. At project flood stage x_3 may increase to 2000 ft for line B and will probably remain about 1500 ft for line C. The greater predicted distance for line B is based on the fact that the thickness of the top stratum along line B averages about 18 ft, whereas it is only 12 to 15 ft at line C.

397. Thickness and permeability of substratum sands. The pervious substratum at Upper Francis consists of a deep stratum of pervious sand with some gravel having an effective thickness of about 125 ft. The gradations of typical foundation sands at the site are plotted on plate 140.

398. The permeability of the pervious substratum was estimated from laboratory permeability tests and correlation of D_{10} vs k_f as shown by fig. 17. These estimates are given in table 18. A value of $k_f = 1400 \times 10^{-4}$ cm per sec was selected as being the best estimate of k_f at the site.

399. Thickness and permeability of top stratum. The top stratum landward of the levee at this site consists essentially of a continuous clay layer approximately 10 to 20 ft thick overlain by natural levee deposits of sandy silt 2 to 7 ft thick for a distance of approximately 1000 ft landward of the seepage berm toe. The effective thickness of top stratum used in the seepage analyses made at the two piezometer lines is given in table 18. The permeability of the clay top stratum as computed from piezometric data obtained at the crest of the 1950 high water and the effective thicknesses as shown in table 18 was about 1×10^{-4} cm per sec. The fact that the permeability of the clay blanket is very much greater than normally would be associated with such soil is attributed to numerous crayfish holes and other fissures in the top stratum. In this connection it is pointed out, however, that k_{bL} at Upper Francis is approximately 2 to 5 times less than that for the thinner clay blanket at the Trotters 54 site.

400. Permeability ratio. The ratio of permeability of the foundation to that of the top stratum is estimated to have been about 1200 along piezometer lines B and C at the crest of the 1950 high water.

401. Seepage flow. Seepage passing beneath the levee at the crest of the 1950 high water, and for the project flood, was estimated using the corresponding measured values of k_f, s, and x_3 for these floods. At the 1950 crest the natural seepage passing beneath the levee at the Upper Francis site was computed to be about 75 gpm per 100 ft of levee, or $Q_s/H = 8$ gpm. Relatively little seepage was observed emerging in the immediate vicinity of the seepage berm at the 1950 crest. As may be noted from the slope of the hydraulic grade line for piezometer B on plate 137, seepage landward of the levee is dispersed over a wide area and some no doubt emerges as far as a mile landward of the levee.

402. Landside substratum pressures. The hydrostatic pressures that developed along the toe of the berm at the crest of the 1950 high water are shown on plate 137. Hydrostatic heads at and landward of the seepage berm are also shown on plate 137. Readings of selected piezometers at or near the landside toe of the levee vs river stages are plotted on plates 138 and 139. Estimated piezometer readings for river stages

higher than those experienced to date also are shown. The head on the levee, top stratum characteristics, substratum pressures, and gradient through the top stratum at certain typical piezometers landward of the levee are given in table 19. Plates 135 and 136 show that changes in rising river stages were quickly reflected in readings from most piezometers landward of the levee. During the 1950 high water there was an estimated lag of approximately 7 days in the development of excess head landward of the levee after the river reached the levee. This is attributed to filling of natural ground storage landward of the levee as the river rose.

403. Uplift pressures that developed during the 1950 high water at the Upper Francis site were not great enough to cause sand boils landward of the levee. However, on the basis of plots shown on plates 138 and 139 and data in table 19, it appears that critical uplift pressures will develop landward of the levee when H is higher than approximately 20 ft. At project flood stage H will be approximately 25 to 28 ft.

404. Excess heads landward of the levee at the crest of the 1950 high water ranged from about 2 to 5 ft, or about 25 to 50% H. Excess heads of 10 to 15 ft probably will have to develop at this site before sand boils will occur. Such boils, when they do occur, may be expected to be quite active because of the high pressures required to cause them and concentration of seepage toward the few points of relief.

405. Estimated gradients through the top stratum at the crest of the 1950 high water ranged from about 0.1 to 0.3. The fact that no sand boils occurred at this site may be attributed largely to the considerable distance to effective seepage entry and the thickness of the top stratum.

Evaluation of seepage problem and
recommendations for control measures

406. A river head of approximately 20 ft on the Upper Francis levee, approximately 5 to 8 ft below project flood stage, may be expected to cause the formation of rather concentrated sand boils landward of the levee. Although the levee withstood a head of approximately 18.5 to 20.5 ft during the 1937 high water without any serious seepage or sand boils being reported, such a stage is about equal to that estimated as being

Table 19

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Upper Francis, Miss., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (168.3): H ft	h_o ft	$\frac{h_o}{H}$ %
A-G	A-1	161.5	-----	12.0	7.0[e]	19.0	17.5	14.9	6.8	2.0	30
B-G	B-4	161.0	-----	13.5	9.5[e]	23.0	18.0	15.3	7.3	1.5	21
B	B-5	160.5	-----	15.0	11.5	26.5	20.0	17.0	7.8	1.8	23
B	B-7	157.0	-----	18.0	0.0	18.0	18.0	15.3	11.3	4.0	35
C-G	C-4	159.5	-----	10.0	8.0[e]	18.0	14.0	11.9	8.8	4.3	49
C	C-5	158.0	-----	12.0	6.5	18.5	12.0	10.2	10.3	3.2	31
C	C-6	158.0	-----	12.0	6.5	18.5	12.0	10.2	10.3	4.9	48

Piez Line	Piez Number	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (185.0): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
A-G	A-1	----	----	----	0.11	23.5	9.7[a]	41	>23.5
B-G	B-4	----	----	----	0.08	24.0	15.3[c]	64	16.7
B	B-5	----	----	----	0.09	24.5	13.5[a]	55	>24.5
B	B-7	----	----	----	0.22	28.0	15.3[c]	55	20.0
C-G	C-4	----	----	----	0.31	25.5	11.9[c]	47	19.6
C	C-5	----	----	----	0.27	27.0	10.2[c]	38	20.0
C	C-6	----	----	----	0.41	27.0	10.2[c]	38	14.4

a, c See paragraph 143.

e Only silt on top of clay included in computing z_t.

required to cause the development of sand boils. Thus, there is no assurance that an additional 2 to 3 ft of net head might not have activated rather severe boils. The maximum river stage experienced during 1950 was not very high and piezometer readings were relatively low. However, on the basis of higher stages and corresponding high substratum pressures at sites similar to Upper Francis (i.e., L'Argent and Baton Rouge), high and probably critical substratum pressures may be expected at Upper Francis at river stages near project flood stages; thus, some additional underseepage control measures are indicated. Filling the riverside borrow pits would result in little reduction in landward pressures; extension of the existing berm would be a possible solution, but is not considered practicable because of the flatness of the hydraulic gradient. Relief wells spaced fairly far apart are recommended to reduce subsurface excess pressures to safe values.

407. The existing seepage berm probably is not sufficiently wide to prevent the formation of critical uplift pressures landward of the berm toe at a project flood stage; however, it is more than adequately thick for its width as illustrated by the gradients shown on plate 137.

408. The narrow sublevee basin from sta 50 to 60 has little effect on the development of critical excess pressures landward of the levee as shown by the hydraulic gradients observed during the 1950 high water. When filled with water, the basin will have a somewhat greater effect on shallow seepage through the natural levee (sandy silt) deposits immediately beneath the levee. However, as the levee and berm have a creep ratio of 18 for a project flood without water in the sublevee basin, the basin is not considered necessary for the control of shallow seepage.

Lower Francis, Mississippi

409. The Lower Francis site was selected for study because of its record of heavy underseepage, and the existence of a relatively uniform thin landside top stratum, and of riverside borrow pits excavated to sand. Investigation of the effectiveness of a wide, pervious berm for controlling underseepage was also desired.

Description of site

410. This site, located immediately downstream of the Upper Francis site, extends from levee sta 110 to 150. The levee is approximately 1500 ft from the Mississippi River and has a net height of approximately 29 ft. Plans of the site, river, borrow pits, surface geology, topography, and piezometer layout are shown on plates 141 and 142; plate 143 is an aerial mosaic of the site. Along the portion of levee studied, riverside borrow pits 10 to 15 ft deep and approximately 700 ft wide have been excavated 5 to 10 ft into the pervious substratum (see plates 142, 144, and 145). The large clay-filled channel previously described for the Upper Francis site lies adjacent to and landward of the Lower Francis site as shown on plates 141 and 142.

411. Prior to 1944, the levee at Lower Francis was located as shown on plate 141. As described on this plate, the former levee was also subject to very heavy underseepage during periods of very high water. At the time the levee was set back at Lower Francis, a large sand seepage berm was provided as shown on plates 141 and 142; this berm is approximately 13 ft thick at the levee toe and 200 ft wide. Sections of the berm are shown on plates 144 and 145. An approximate relation between river stages at Lower Francis and the river gage at Helena, Ark., is shown on plate 140.

412. History of underseepage. The levee was set back to its present position in 1944, and the first high water against it occurred in 1945. During this high water, an H of about 16 ft caused very heavy seepage and numerous sand boils. Six large sand boils developed between sta 115 and 128, and 54 sand boils occurred within 200 ft of the toe of the berm between sta 135 and 151. Upstream from sta 135, seepage emerged at the surface of the berm along the landward half; downstream from sta 135, the toe of the berm was unstable. As the toe of the berm was built of clay and served as a retaining dike while the remainder of the berm was being dredged in place, it is believed that the clay toe was inhibiting the emergence of seepage from the berm. During the 1950 high water, an H of approximately 13 ft caused medium to heavy underseepage between sta 105 and 150 with numerous sand boils from sta 141 to 147. The sand

boils first appeared about 5 February 1950, and by 8 February were estimated to be flowing at a rate of 5 to 50 gpm each; some boils discharged as much as 1 cu yd of sand. The locations of the seepage and sand boil areas are shown on plate 142. The seepage along the toe of the berm from sta 140 to 147 was very heavy; in fact, some places along the berm toe were so soft that it was impossible to walk across the area. The topography at the Lower Francis site is such that seepage measurements could not be made.

413. Piezometer installation. In 1948 two lines (C and E) of piezometers were installed perpendicular to the levee at sta 130 and 145. Piezometer line C was located where soil and geological conditions are most typical of the site. Line E was located where geological conditions are such as to create a particularly dangerous underseepage condition (see plate 142). Two line C piezometers were installed in the riverside borrow pits to measure the head in the sand foundation immediately riverward of the levee. Piezometers were also installed along the toe of the seepage berm from sta 115 to 145 (line H). The tips of most of the piezometers were installed immediately beneath the clay top stratum. Piezometer readings were obtained during the 1950 high water.

Geology of site and soil conditions

414. The general geology of the site is illustrated on plate 141; the type and thickness of top stratum deposits are shown in more detail on plate 142. The site is situated on point bar deposits laid down as the river gradually enlarged a meander loop leading eventually to a cutoff at the end of stage 11 (plate 141). The top stratum landward of the levee consists of a relatively thin blanket of clay underlain by silt which gradually increases in thickness until it finally merges with the thick clay and silt deposits filling the cutoff channel of course 11 (see plates 142, 144, and 145). The clay-filled channel has a crescent outline and at the center of the site lies approximately 2000 ft landward of the levee. The near edge of this old channel crosses under the toe of the seepage berm at approximately sta 145. The clay and silt deposits of the old channel range in thickness from 20 to 30 ft (plate 144).

415. Relation of underseepage to geology. The primary geological

feature affecting seepage at the site is the massive clay-filled channel landward of the levee. As may be noted from plate 142, all of the observed seepage and sand boils at the site occurred in the area between the levee and the old channel filling. The removal of the natural top blanket riverward of the levee has also aggravated the underseepage problem significantly. The fact that most of the seepage landward of the levee was dispersed rather than concentrated in the form of sand boils can be attributed to the relatively uniform nature and thinness of the top stratum landward of the levee for a distance of 300 or 400 ft except where the clay-filled channel crossing beneath the levee at sta 145 concentrated the seepage.

416. Soil profile and piezometer lines. The locations of piezometers and borings are shown on plates 142 and 143. Soil profiles and piezometer lines, both perpendicular and parallel to the landside toe of the levee, are shown on plates 144 through 147.

417. The sediments making up the top stratum in the point bar area are relatively uniform in type and thickness (plates 142, and 144 to 146). In general, the top stratum landward of the seepage berm for 500 ft or more consists of about 4 to 6 ft of clay underlain by 1 to 2 ft of sandy silts, except in the area between sta 140 to 148 where the clay top stratum is somewhat thicker.

418. The pervious substratum at the site consists of an upper stratum of fine to medium sand approximately 20 ft thick which is underlain by about 130 ft of very pervious, medium to coarse sands with some gravel. This pervious stratum no doubt extends to the bank of the Mississippi River, a distance of approximately 1200 ft from the center line of the levee.

Analysis of piezometric and seepage data

419. River stages and piezometer readings observed at the site during the 1950 high water are plotted on plates 148 and 149. At the crest of this high water, the head on the levee was about 13 to 14 ft. Piezometric gradients existing in the pervious substratum beneath the levee at piezometer lines C and E are shown on plate 150. The hydrostatic

head along the toe of the berm as measured by the piezometers along line H is also shown on this plate. Excess heads of about 2 to 3 ft developed along the berm toe from about sta 115 to 145. Some excess hydrostatic head existed as far as 600 to 800 ft landward of the levee; the gradient beyond this point was practically flat. Based on the gradient curves on plate 150, all seepage passing beneath the levee apparently emerged between the levee and the filled channel of course 11. The highest excess head at the toe of the berm during the 1950 high water was about 3 ft.

420. A summary of information pertaining to the site and the results of analyses of piezometric and seepage data based on borrow pit conditions and seepage control measures as existing in 1950 are given in table 20.

Table 20

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Lower Francis, Miss., Site

Factor	Line C		Line E		Design Values		
	1950 Flood	Project Flood	1950 Flood	Project Flood	Sta 114-122	Sta 122-138	Sta 138-147
River stage (crest)	167.6	182.6	167.6	182.6	182.6	182.6	182.6
Average el of ground or tailwater	154.0	-----	154.0	-----	155.5	154.0	154.0
Head on levee (H)	13.6	28.6	13.6	28.6	27.1	28.6	28.6
Piezometers used in analysis	C-4 & C-6	-----	E-1 & E-2	-----	-----	-----	-----
Riverside borrow pit, width, ft	600	-----	600	-----	700	600	600
Top stratum	0-4 ft clay	-----	0-4 ft clay	-----	-----	-----	-----
			0-2 ft silt				
Average stratum	Sand	-----	1 ft clay	-----	Sand	Sand	1 ft clay
Distance from riverside levee toe to river (L_1)	1100	-----	1300	-----	1500	1100	1300
Base width of levee (L_2)	575	-----	575	-----	575	-----	-----
Landward extent of top stratum (L_3)	850	-----	600	-----	1000	800	600
Distance to effective seepage source (s)	1010	950	1280	1100	900	1000	1100
Effective length of riverside blanket (x_1)	435	375	705	525	325	425	525
Distance to effective seepage exit (x_3)	250	200	460	250	330	370	610
Effective thickness of sand substratum (d)	135	-----	135	-----	135	-----	-----
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1600	-----	1600	-----	1600	-----	-----
Laboratory permeability tests	1100	-----	1100	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	2300	-----	2500	-----	-----	-----	-----
Seepage and piezometric data	-----	-----	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	-----	-----	-----	-----	-----
Top stratum, type	Clay	-----	-----	-----	Clay	-----	-----
Effective thickness for seepage analysis (z_{bL})	5.0	-----	7.5	-----	4.0	5.0	7.0
Critical thickness (z_t)	4.5	-----	6.5	-----	3.5	4.5	6.0
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	17	27	10	26	-----	-----	-----
Piezometric data and blanket formulas	17	27	10	26	-----	-----	-----
Piezometric data and seepage measurements	-----	-----	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	93	60	165	60	200	200	300
Blanket formula	93	59	166	62	-----	-----	-----
Natural seepage measurements	-----	-----	-----	-----	-----	-----	-----
Natural seepage beneath levee							
Q_s, gpm/100 ft of levee	340	790	250	675	-----	-----	-----
Q_s/H, gpm/ft of head/ 100 ft of levee	25.2	27.6	18.3	23.6	-----	-----	-----

421. Source of seepage. Seepage may enter the pervious aquifer through the bank and bed of the Mississippi River and in the riverside borrow pits which extend 8 to 10 ft into the pervious substratum (plates 144 and 145).

422. Distances to the point of effective seepage entry were determined at piezometer lines C and E for various days during the 1950 high water period and are plotted in fig. 37. The values of s shown in fig. 37 and on plate 150 indicate that most of the seepage enters the substratum sands through the borrow pits wherein the substratum sand has been exposed. The values of s as measured at lines C and E at the crest of the 1950 high water were about 1000 and 1280 ft, respectively. These values are equivalent to an open seepage entry face only 400 to 700 ft from the riverside toe of the levee. Values for s at the project flood are estimated to be 950 and 1100 ft for piezometer lines C and E, respectively.

423. Seepage exit. Values of x_3 vs river stage are plotted in fig. 37. The average ground surface and tailwater elevation used in determining x_3 was 154.0 as shown in table 20 and fig. 37. (The area below el 154.0 was submerged in 1950.) At piezometer line C where the top stratum consists of about 5 ft of clay, x_3 was about 250 ft during

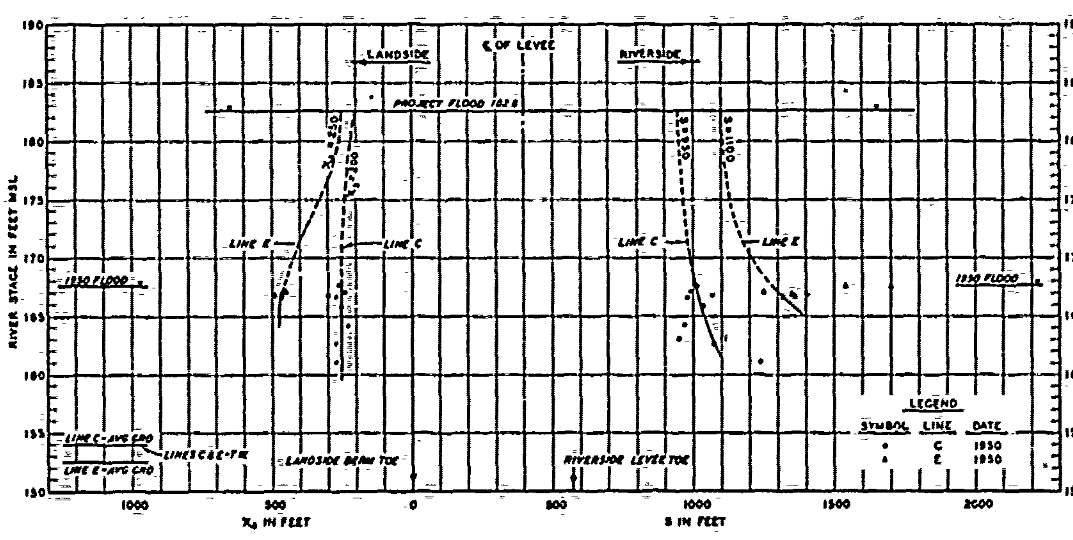

Fig. 37. Distances to effective seepage source and exit.
Lower Francis, lines C and E

the 1950 high water. At piezometer line E, where the top stratum consists of about 7.5 ft of clay, x_3 was about 460 ft near the crest of this high water. The greater value of x_3 at line E is attributed to the thicker clay top stratum at this location as compared to that at line C. As the values of x_3 are somewhat less than the respective distances L_3 to the channel fill landward of the piezometer lines, it can be concluded that the channel fill does not materially affect the substratum pressures, although the channel has an influence on the distribution of seepage landward of the levee. Values of x_3 estimated for the crest of the project flood are 200 and 250 ft for lines C and E, respectively (see table 20 and fig. 37).

424. Thickness and permeability of substratum sand. The pervious foundation has an effective thickness of about 135 ft. The permeability of the sand aquifer was estimated from laboratory permeability tests made on samples of sand obtained with a bailer and correlation of D_{10} vs k_f (fig. 17). The results of these determinations are given in table 20 as is the permeability believed to be most representative of the sand aquifer. (Grain size curves for typical substratum sands are shown on plate [illegible].) From these data the average permeability of the substratum sands is considered to be about 1600×10^{-4} cm per sec.

425. Thickness and permeability of top stratum. The average top stratum landward of the berm toe is primarily clay and is considered to have an effective thickness of about 3 to 7.5 ft (plates 144 through 147). The top stratum is relatively uniform landward of the levee toe, but farther landward becomes considerably thicker. On the basis of the thicknesses in table 20, k_{bL} computed from piezometric data during the 1950 high water was about 17 and 10×10^{-4} cm per sec at lines C and E, respectively. The lower permeability at line E can probably be attributed to the somewhat greater thickness of clay in the vicinity of this line.

426. Permeability ratio. The ratio of the permeability of the foundation to that of the top stratum at piezometer lines C and E is estimated to be about 95 and 165, respectively. Estimates of k_f/k_{bL} for the crests of the 1950 and project floods are given in table 20.

427. Seepage flow. The natural seepage passing beneath the levee

at the crest of the 1950 high water, and (estimated) for the project flood, at lines C and E was computed from corresponding values of H, s, and x_3. The value of Q_s at the 1950 crest ranged from about 250 to 340 gpm per 100 ft of levee (Q_s'/H = 18 to 25 gpm) with the greater seepage estimated at line C. Natural seepage at the project flood is estimated at about 700 to 800 gpm per 100 ft of levee, or Q_s/H = 25 gpm. The greater seepage at line C is believed to result from the thinner top stratum landward of the levee which offers less resistance to seepage flow than that at line E. From the above data, it is concluded that the Lower Francis site is subject to a very high rate of natural seepage.

428. Landside substratum pressures. Hydrostatic pressures that developed along the toe of the berm at or near the crest of the 1950 flood are shown on plate 150 (line H). Readings of selected piezometers at or near the landside toe of the levee vs river stages are plotted on plates 151 and 152. Also shown are the estimated substratum pressures for flood stages up to the project flood, and computed maximum piezometer readings based on i_c = 0.85. The head on the levee, top stratum characteristics, and substratum pressures at certain typical piezometers along the landside toe of the berm are given in table 21. Plates 148 and 149 show that in general most of the piezometers followed rising river stages with little lag. Uplift pressures sufficient to cause sand boils and to make the top stratum become quick developed along the entire Lower Francis site at H = 12 ft during the 1950 high water. The uplift pressures beneath the top stratum ranged from about 8 to 25% H during 1950 and

Table 21

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Lower Francis, Miss., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (167.6): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (182.6): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
A-H	A-1	156.0	-----	3.0	0.0	3.0	3.0	2.6	11.6	1.6	14	----	0.53	----	----	26.6	1.6[b]	6	9.8
H	B-1	154.5	-----	3.2	0.0	3.2	3.2	2.8	13.1	3.0	23	----	0.93	----	----	28.1	3.0[b]	11	11.6
C-H	C-7	154.5	-----	4.0	2.0	6.0	4.5	3.8	13.1	1.7	13	----	0.38	----	----	28.1	3.8[c]	14	18.5
H	D-1	153.8	154.0	5.0	2.0	7.0	5.0	4.2	13.1	2.5	19	----	0.50	----	----	28.1	2.5[b]	9	12.0
E-H	E-2	154.2	-----	3.5	2.7[a]	6.2	6.2	5.3	13.1	3.2	24	----	0.52	----	----	28.1	3.4[b]	12	12.9
E	E-3	151.5	154.0	7.5	0.0	7.5	7.5	6.4	13.1	1.1	8	0.15	----	----	----	28.1	1.1[b]	4	11.8

b, c See paragraph 143.

a Sand on top of clay.

resulted in gradients of about 0.4 to 0.9. In general, heads beneath the top stratum during the project flood stage probably will not be greater than those experienced in 1950 (about 2 to 3 ft) as the latter are generally representative of the critical heads that can develop.

Evaluation of seepage problem and recommendations for control measures

429. H of about 11 to 13 ft during the 1950 high water and 16 ft during the 1945 high water caused numerous sand boils along the Lower Francis site. An analysis of the piezometric data and soil conditions at the site indicates that uplift pressures in excess of those already experienced will not develop and that river stages exceeding the 1950 high water crest will tend only to increase the number and size of sand boils. The seepage problem is made more severe by the presence of the filled channel from sta 140 to 150 landward of the levee. Although the seepage berm improves the safety of the levee in that it will prevent sloughing of the landside levee slope and will force the point of any subsurface piping farther landward, the width of the berm is not adequate to prevent development of sand boils.

430. Because of the relatively low maximum excess pressures that can exist due to the thin top stratum, and because of the high creep ratio of 21, there is some question regarding the extent of additional control measures required. It is believed that a line of relief wells should be installed along the toe of the berm from sta 140 to 150 where the clay-filled channel is close to the levee and tends to concentrate seepage and sand boils along this reach of levee. Upstream from sta 140, the landside berm should be widened. The existing berm is thicker than required and part of the berm material could be used for widening if desired.

431. At no time during the 1950 high water was any excess hydrostatic head observed above the surface of the seepage berm and the predicted gradient beneath the levee and berm at line C for the project flood stage is below the surface of the berm. Even though the seepage berm is composed of sand, its thickness prevents the emergence of seepage at the berm surface and as a result the berm acts essentially as an impervious berm.

Bolivar, Mississippi

432. Bolivar was selected as a site for study because it had been subject to underseepage and sand boils during previous high waters and was considered representative of a medium-thick, uniform clay landside top stratum. It also provided an opportunity for studying the effectiveness of sublevees as a seepage control measure.

Description of site

433. The site is located along the east bank levee of the Mississippi River approximately 2 miles northwest of Benoit, Miss., and extends from levee sta 2190 to 2210. Plans of the site, borrow pits, surface geology, topography, and piezometers are shown on plates 153 and 154; plate 155 is an aerial mosaic of the site. At this site the levee is approximately 8 miles from the main channel of the Mississippi River; however, it is only about 1200 to 1500 ft from Bolivar Chute, which lies riverward of the levee. Riverside borrow pits 5 to 10 ft deep and about 300 ft wide have been excavated along most of the site. Sublevees extend along the levee from approximately sta 2122 to 2253 (plate 153). No seepage berms have been constructed at this site. The levee has a net height of approximately 26 ft. An approximate relation between river stages at Bolivar and the river gage at Arkansas City, Ark., is shown on plate 166.

434. History of underseepage. Although considerable seepage probably occurred along this site during the 1937 high water, there is no record of it. Heavy underseepage and numerous pin boils were reported between sta 2020 and 2250 during the high water of 1945, and the sublevee basins were between one-fourth and one-half filled with seepage. H during this high water was about 11 ft.

435. During the 1950 high water, when H = 9 ft, the sublevee basin became filled with water, probably as a result of both surface runoff and seepage. It was impossible to determine whether any sand boils developed in the sublevee basin; however, very little, if any, seepage occurred between the levee toe and the water in the basin.

436. Piezometer installation. In 1948 a long line (D) and two

shorter lines (B and G) of piezometers were installed perpendicular to the levee at sta 2200, 2192, and 2210, respectively (plate 154). One piezometer (D-1) was installed on line D, in the riverside borrow pit. Additional piezometers (line K) were installed along the landside toe of the levee from approximately sta 2190 to 2210, one of which (A-1) was located at a point considered particularly critical as regards seepage. Piezometer readings were obtained during the high water of 1950.

Geology of site and soil conditions

437. The general geology of the site is illustrated on plate 153; the type and thickness of top stratum materials are shown in more detail on plate 154. Two very different geological conditions exist at the site. It is believed that an ancient channel (course H), now filled with thick deposits of silt and clay, at one time crossed the site at the levee. Subsequent to the filling of this old channel the river cut out a part of the old filling at the center of the site, leaving point bar deposits as it migrated from course 3 to course 5. Thick clays deposited in the abandoned channel of course H lie immediately landward of the levee from sta 2190 to 2203 (see plate 154). As shown on this plate, relatively narrow and widely spaced swales cross the levee at an angle of approximately 90° downstream of sta 2190. The filling in former course H consists of approximately 35 ft of silts and clays. The top stratum along the toe of the levee from sta 2190 to 2206 consists of clay 8 to 10 ft thick except where swales approximately 15 ft thick pass beneath the levee. Downstream of sta 2206 the top stratum also consists of clay, but it is only about 7 ft thick and underlain by approximately 4 ft of silty sands and sandy silt (see plate 154 and sections K and L on plates 158 and 159). About 3 to 5 ft of clayey and silty natural levee deposits blanket the site.

438. Relation of underseepage to geology. The recorded seepage data are not adequate for drawing conclusions regarding the effect of distribution of sediments on the location and formation of serious underseepage. However, a potentially serious underseepage condition exists between the toe of the levee and the massive clay-filled channel from sta 2190 to 2200, particularly in the area bounded by piezometers A-1, B-1, and B-2. The reason that more serious underseepage has not occurred

at this site is probably because t. [illegible]erside borrow pits are blanketed by 2 to 5 ft of clay. A reason the site is potentially dangerous is that the top stratum immediately landward of the levee is 8 to 10 ft thick -- thick enough to permit the formatic[illegible] considerable excess hydrostatic pressure but not thick enough to withstand 50 per cent of the possible head on the levee. Where such conditions exist, sand boils usually develop in localized areas and because they are few in number may become very active and result in serious piping, such as occurred at the Stovall site previously discussed.

439. The top stratum at piezometer line G consists essentially of a uniform stratum of clay about 5 ft thick similar to the top stratum at Lower Francis (see plate 157).

440. Soil profiles and piezometer lines. The locations of piezometers and borings are shown in plan on plate 154; soil profiles and piezometer lines, both perpendicular and parallel to the landside toe of the levee, are shown on plates 156-159. Piezometer line B was located at a point where the most critical seepage condition was thought to exist (plate 154). Piezometer line D was located at a point where the top stratum was fairly uniform as regards both thickness (8 to 12 ft) and type for a distance of 800 ft landward of the levee. Piezometers were located on lines B and D adjacent to a drainage ditch landward of the levee which significantly reduces the top stratum thickness, particularly at boring 18 (plates 154 and 156). Piezometer line G was located in an area of uniform top stratum consisting of approximately 4 to 6 ft of clay underlain by several feet of sandy silt. Additional piezometers were located along the toe of the levee (line K) as shown on plate 154. The tips of most of the piezometers were located immediately beneath the clay top stratum.

441. The pervious substratum at the site consists of an upper stratum of fine to very fine sand approximately 10 ft thick underlain by approximately 30 ft of fine to medium sands which in turn are underlain by approximately 75 ft of alternating strata of medium to coarse sands.

Analysis of piezometric and seepage data

442. River stages and piezometer readings observed at the site during the 1950 high water are plotted on plates 160 and 161. At the crest of this high water, H was about 6.5 ft. Piezometric gradients existing in the pervious substratum beneath the levee along piezometer lines B, D, and G perpendicular to the levee are shown for selected river stages on plates 162-164. Hydrostatic heads along the toe of the levee as measured by piezometer line K are plotted on plate 164, and ranged from about 1.0 to 2.5 ft above the elevation of water in the sublevee basin at the 1950 crest. Excess heads above the ground surface existed only a short distance landward of the sublevee because of the low head on the levee. A summary of information pertaining to the site and the results of analyses of piezometric and seepage data subsequently discussed are given in table 22.

443. Source of seepage. Seepage may enter the pervious foundation in Bolivar Chute and in the other river channel, shown as (19) on plate 153, and through riverside borrow pits (see plate 163).

444. Values of s at piezometer lines G and B during the 1950 high water are plotted in fig. 38. The values of s shown in fig. 38 and plates 163 and 164 indicate that seepage enters the sand substratum primarily through borrow pits immediately riverward of the levee. Most of the natural, impervious top stratum has been removed in the borrow pits at piezometer lines D and G; some thin strata of silty sands and sandy silts still remain in the bottoms of these pits and about 2 ft of clay apparently remain in the bottom of the pits at line B. Clean sands exist immediately beneath these layers. At piezometer lines D and G, s was about 600 to 650 ft at the crest of the 1950 high water (fig. 38 and table 22). It may be seen in fig. 38 that s decreased considerably as the river rose. Based on piezometer data obtained during the 1950 high water, s may decrease to as little as 500 to 550 ft at project flood stage if borrow pit conditions are not altered. Thus the effective source of seepage entry would be only about 200 ft from the riverside toe of the levee at such a flood stage.

Table 22

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Bolivar, Miss., Site

Factor	Line B 1950 Flood	Line B Project Flood	Line D 1950 Flood	Line D Project Flood	Line G 1950 Flood	Line G Project Flood	Design Values Sta 2190-2205	Design Values Sta 2205-2220
River stage (crest)	147.5	167.2	147.5	167.2	147.5	167.2	167.2	167.2
Avg tailwater el in sublevee basin	141.0	141.0	141.0	141.0	141.0	141.0	141.0	141.0
Head on levee (H)	6.5	26.2	6.5	26.2	6.5	26.2	26.2	26.2
Average ground elevation	139.0	-----	139.0	-----	140.0	-----	139.0	140.0
Piezometers used in analysis	-----	-----	D-2, -3*	-----	G-1, -2*	-----	-----	-----
Riverside borrow pit, width, ft	300	-----	300	-----	300	-----	300	300
Top stratum	2 ft clay	-----	0-3 ft Cl 2-5 ft Sl Sd	-----	2-4 ft Cl 3 ft Fl Sd	-----	-----	-----
Average stratum	2 ft clay	-----	5 ft Sl Sd	-----	5 ft Sd Sl	-----	2 ft clay	5 ft Sd Sl
Distance from riverside levee toe to river (L_1)	1500**	-----	1500*	-----	1500*	-----	1500*	1500*
Base width of levee (L_2)	330	-----	330	-----	330	-----	330	330
Landward extent of top stratum (L_3)	160	-----	900	-----	∞	-----	500	∞
Distance to effective seepage source (s)	-----	-----	650	550	600	500	500	500
Effective length of riverside blanket (x_1)	-----	-----	320	220	270	170	-----	-----
Distance to effective seepage exit (x_3)	-----	-----	365	250	320	175	350	300
Effective thickness of sand substratum (d)	90	-----	90	-----	90	-----	90	90
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1200	-----	1200	-----	1200	-----	1200	1200
Laboratory permeability tests	-----	-----	-----	-----	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	1310	-----	1310	-----	1310	-----	-----	-----
Seepage and piezometric data	-----	-----	-----	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	-----	-----	-----	-----	-----	-----
Top stratum, type	Clay	-----	Clay	-----	Clay	-----	Clay	Clay
Effective thickness for seepage analysis (z_{bL})	7.0	-----	13.0	-----	6.0	-----	7.0	6.0
Critical thickness (z_t)	7.0	-----	13.0	-----	6.0	-----	7.0	6.0
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	-----	-----	10	22	6	21	-----	-----
Piezometric data and blanket formulas	-----	-----	10	22	6	21	-----	-----
Piezometric data and seepage measurements	-----	-----	-----	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	-----	-----	120	54	200	57	165	165
Blanket formula	-----	-----	120	54	200	57	-----	-----
Natural seepage measurements	-----	-----	-----	-----	-----	-----	-----	-----
Natural seepage beneath levee								
Q_s, gpm/100 ft of levee	-----	-----	101	520	112	620	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	-----	-----	15.6	19.8	17.3	23.6	-----	-----

* Heads at D-3 and G-2 were increased to obtain average head in sand substratum as described in paragraph 132.
** Bolivar Chute.

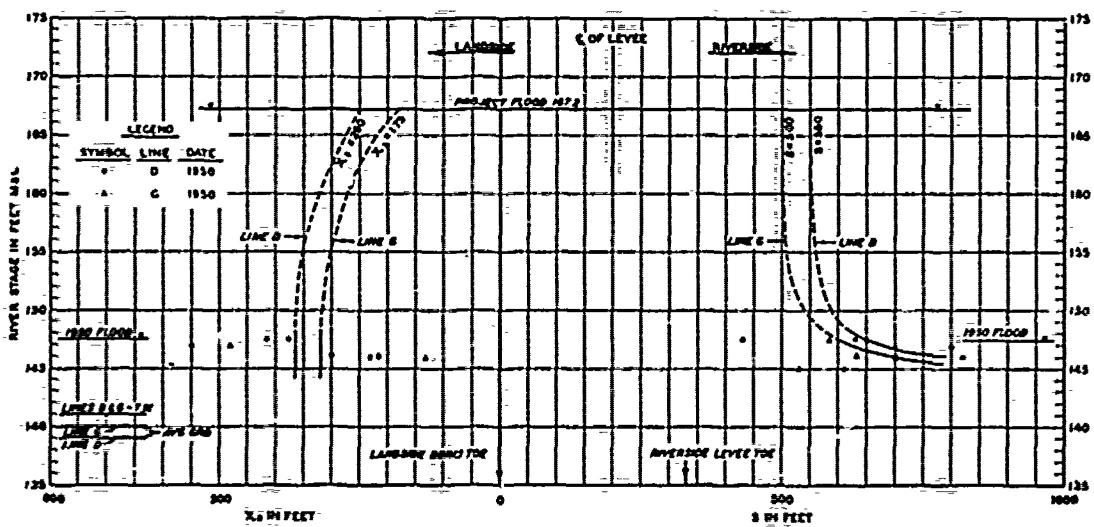

Fig. 38. Distances to effective seepage source and exit. Bolivar, lines D and G

445. Seepage exit. Values of x_3 vs corresponding river stages are plotted in fig. 38 for the 1950 high water. There is a great deal of scatter in the values of x_3 at both piezometer lines D and G. The average x_3 at the 1950 crest was about 350 ft. On the basis of the maximum computed hydrostatic head that can exist landward of the levee at project flood stage it is believed that x_3 might decrease to 200 or 250 ft for such a flood stage. The short x_3's at the Bolivar site may be attributed to the relatively thin top stratum immediately landward of the levee.

446. Thickness and permeability of substratum sands. The pervious substratum at the Bolivar site consists of a deep stratum of pervious sands with some gravel in the lower third of the aquifer; it is considered to have an effective thickness of about 90 ft. The gradations of typical foundation sands at the site are plotted on plate 166. The permeability of the pervious substratum was estimated from a correlation of D_{10} vs k_f as shown in fig. 17, and laboratory permeability tests on remolded samples. A $k_f = 1200 \times 10^{-4}$ cm per sec was estimated for the aquifer at the site.

447. Thickness and permeability of top stratum. The top stratum landward of the levee varies considerably in thickness and uniformity, as illustrated on plates 154, 156, and 157. Except for the thick clay-filled channel (H), the top stratum from sta 2190 to 2205 consists of clay approximately 13 ft thick. Downstream of sta 2205, the top stratum appears to be considerably thinner, consisting only of about 6 ft of clay. The effective thickness of top stratum used in the seepage analyses made at piezometer lines D and G is given in table 22. The permeability of the clay stratum, as computed from piezometric data obtained at the crest of the 1950 high water and the effective thickness as shown in table 22, was about 6 to 10 x 10^{-4} cm per sec.

448. Permeability ratio. The ratio of permeability of the foundation to that of the top stratum along piezometer lines D and G at the crest of the 1950 high water is estimated to have been about 100 and 200, respectively (table 22).

449. Seepage flow. Seepage passing beneath the levee at the crest of the 1950 high water, and for the project flood, was estimated using

the corresponding measured values of H , s , and x_3 for these floods (see table 22). At the 1950 crest, the natural seepage passing beneath the levee was computed to be about 100 gpm per 100 ft of levee, or Q_s/H = 16 gpm. At project flood stage, seepage beneath the levee may amount to as much as 500 to 600 gpm per 100 ft of levee, or Q_s/H = about 20 gpm. Because of water in the sublevee basin landward of the levee, no idea of the rate of seepage into the inclosed area could be obtained. On the basis of the hydraulic grade line shown on plates 163 and 164 for the 1950 crest, it appears that 70 to 90 per cent of the seepage passing beneath the levee was emerging either in the sublevee basin or immediately landward thereof.

450. Landside substratum pressures. Hydrostatic pressures that developed along the levee toe (piezometer line K) at the crest of the 1950 high water are shown on plate 164. Hydrostatic heads in the sublevee basin and landward thereof at the 1950 crest are shown on plates 162-164. Readings of selected piezometers at or landward of the levee toe vs river stages are plotted on plate 165. The head on the levee, top stratum characteristics, substratum pressures, and the gradient through the top stratum at certain typical piezometers are given in table 23.

Table 23

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Bolivar, Miss., Site

Piez Line	Piez Number	Avg Gradient at Piez, el. ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c (0.85 z_t) ft	Crest of 1950 Flood (147.5): H ft	h_o ft	h_o/H %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (167.2): H ft	h_o ft	h_o/H %	Est H at i_c ft
K	A-1	140.0	141.0	16.0	0.0	16.0	16.0	13.6	6.5	1.1	17	----	----	----	0.07	25.2	----	--	----
B-K	B-1	139.0	141.0	8.0	2.0	10.0	8.0	6.8	6.5	2.2	34	----	----	----	0.28	26.2	6.8[c]	26	10.1
B-L	B-2	138.0	141.0	7.0	0.0	7.0	7.0	6.0	6.5	0.0	0	----	----	----	0.0	26.2	----	--	----
K	C-1	139.0	141.0	11.0	0.0	11.0	11.0	9.4	6.5	3.5	54	----	----	----	0.32	26.2	----	--	----
		137.0[e]	141.0[e]	9.0	0.0	9.0	9.0	7.7	6.5	3.5	54	----	----	----	0.39	26.2	7.7[c]	29	10.4
D-K	D-3	138.5	141.0	10.0	3.0[f]	13.0[f]	13.0	11.0	6.5	2.4	37	----	----	----	0.18	26.2	4.2[d]	16	6.2
D	D-5	141.5	-----	8.0	3.0[f]	11.0[f]	11.0	9.4	6.0	0.2	3	----	----	----	0.02	25.7	----	--	----
		140.0[g]	140.5	8.0	1.5	9.5	9.5	8.1	7.0	1.2	17	----	----	----	0.13	26.7	----	--	----
D	D-7	140.0	-----	----	---	----	----	----	----	---	--	----	----	----	----	----	----	--	----
		135.0[g]	136.0	11.0	0.0	11.0	11.0	9.4	11.5	4.3	37	----	----	----	0.39	31.2	9.4[c]	30	17.2
K	E-1	141.0	141.0	9.0	3.0[f]	12.0	12.0[f]	10.2	6.5	2.8	43	----	----	----	0.23	26.2	----	--	----
K	F-1	140.5	141.0	5.2	5.0	10.2	7.0	6.0	6.5	1.7	26	----	----	----	0.24	26.2	----	--	----
		136.5[e]	141.0	1.0	5.0	6.0	6.0	5.1	6.5	1.7	26	----	----	----	0.28	26.2	1.8[d]	7	5.6
G-K	G-2	140.0	141.0	5.0	3.0	8.0	7.0	6.0	6.5	1.8	28	----	----	----	0.26	26.2	----	--	----
G-L	G-3	139.0	141.0	4.0	3.0	7.0	6.0	5.1	6.5	1.4	22	----	----	----	0.23	26.2	----	--	----
		136.5[e]	141.0	1.5	3.0	4.5	4.5	3.8	6.5	1.4	22	----	----	----	0.31	26.2	----	--	----

c, d See paragraph 143.
e Adjacent landside borrow pit.
f Silt on top of clay.
g Adjacent drainage ditch.

Elevation of water in sublevee basins 2 March 1950 was 141.2.
Elevation of water landward of sublevees 2 March 1950 was 140.4.

451. The data on plates 160 and 161 show that after the ground-water storage was filled the piezometers landward of the levee reacted rapidly to changes in river stage. As at other sites upstream of Bolivar there was an estimated lag of approximately 7 to 10 days in the development of excess heads landward of the levee after the river reached it. This again is attributed to the filling of the natural ground storage landward of the levee as the river rose.

452. Uplift pressures that developed during the 1950 high water at Bolivar were not great enough to create any known sand boils landward of the levee. The fact that no sand boils occurred may be attributed to the low head of water against the levee. On the basis of plots shown on plate 165 and data in table 23, critical uplift pressures will probably develop landward of the levee at river stages higher than approximately 10 ft. (H of 10 to 11 ft during the 1945 high water is reported to have caused heavy underseepage and numerous pin boils.) The project flood stage will create an H of approximately 26 ft with a water surface elevation in the sublevee basins of 141.

453. Excess heads landward of the levee at the crest of the 1950 high water ranged from about 1 to 4 ft, or about 20 to 50% H (table 23). Generally, excess heads of 4 to 7.5 ft would be required for sand boils to occur along this site except possibly in the bottom of the sublevee basin near piezometer F-1. An excess head of 4.3 ft existed above the estimated water-surface elevation in the landside drainage ditch near piezometer D-7. If it were not for the limited amount of excess pressure that can develop immediately landward of the levee, a river stage of 17 ft could be expected to produce sand boils in the bottom of the drainage ditch at piezometer D-7.

454. Estimated gradients through the top stratum at the crest of the 1950 high water were relatively low, ranging from about 0.25 to 0.40.

Evaluation of seepage problem and recommendations for control measures

455. An H of approximately 10 ft, 16 ft below project flood stage, may be expected to cause the formation of sand boils landward of the levee, particularly between sta 2190 and 2192 and in the bottom of

the landside borrow pits from sta 2190 to 2220. Consequently, seepage control measures in addition to the present sublevee basins are indicated. The maximum river stage during the 1950 high water created an H of only 6.5 ft and thus was not high enough to demonstrate visually the need for control measures. No record of seepage observations during the 1937 high water, maximum H of 18 ft, could be found. Although the present sublevee basins will decrease the net head acting on the levee by about 3 ft if they are filled, this reduction will be more than offset by the reduction in the thickness of the landside top stratum inside the subleveed area as the result of borrow operations to construct the sublevees.

456. Raising the sublevees to a height that would insure adequate reduction in net head landward of the levee is considered impracticable. The sublevees could be degraded, the material used to fill the original borrow pits excavated for their construction, and a seepage berm constructed. However, for any practical width of seepage berm a critical condition will remain where the landside toe of the seepage berm intersects the massive clay-filled channel (H) such as now exists between piezometers A-1 and B-1. If a seepage berm is selected as the method for controlling underseepage at the Bolivar site, it probably will still be necessary to install a few wells at the intersection of the landside toe of the berm and former river course (H). Another method suitable for the control of underseepage at this site is a line of relief wells along the landside toe of the present levee. Installation of relief wells along this site would also make possible degrading of the existing sublevees to fill in the existing landside borrow pits and return this area to normal land use.

Eutaw, Mississippi

457. Eutaw was selected as a site for study primarily because it had been subject to heavy underseepage during the 1945 high water and a new large berm had been constructed subsequently which, at the time of selection, had not been subjected to any high water. It was also desired to measure the drop in the hydraulic gradient in the pervious stratum

across a filled channel that extended 30 to 40 ft down into the pervious aquifer, and to determine the increase in head in the pervious foundation beneath the new berm as compared to a point just upstream where geological conditions were similar and no berm exists.

Description of site

458. The site is located along the east bank levee of the Mississippi River approximately one-half mile from the town of Eutaw, Miss., and lies between sta 2835 and 2880. Plans of the site, river, borrow pits, surface geology, topography, and piezometers are shown on plates 167 and 168; plate 169 is an aerial mosaic of the site. Rather extensive borrow pits have been excavated riverward of the levee which, though relatively deep and wide, are presently blanketed by about 5 to 8 ft of silty sand and sandy silt with some clay in spots. The levee has a net height of approximately 31 ft. An approximate relation between river stages at Eutaw and the Mississippi River gage at Arkansas City, Ark., is shown on plate 166.

459. History of underseepage. It is not known whether any underseepage occurred during the 1937 high water when H = 15 ft. However, heavy underseepage extending to a slough about 200 ft landward of the levee toe occurred during the 1945 high water when H was 9.4 ft. Numerous pin boils were observed at the toe of the levee and for a distance of about 80 ft landward between sta 2840 and 2912. In 1947 a large seepage berm approximately 12 ft thick at the levee toe and 200 ft wide was constructed.

460. During the 1950 high water, H = approximately 9 ft, no seepage or sand boils of consequence were reported. The area landward of the levee below contour 132 was under water during this high water.

461. Piezometer installation. In 1948 a line of piezometers (D) was installed perpendicular to the levee at sta 2860 and extends from the riverside borrow pit landward for approximately 1500 ft. Several other piezometers (lines B, F, and J) are located along the toe of the levee and seepage berm as shown on plate 168. The tips of the line D piezometers were installed at several elevations as shown on plate 170. Piezometer readings were obtained during the 1950 high water.

Geology of site and soil conditions

462. The general surface geology at the site is shown on plates 167 and 168. Plate 167 shows the location of former river courses, swales, and natural levee deposits which blanket the area. The character and thickness of the top stratum are shown in more detail on plate 168.

463. The levee and seepage berm are located on old channel filling and channel bar deposits laid down when the river shifted its course during former stages 10 to 13 (plate 167). Although stage 13 was followed by a cutoff, the channel was subsequently filled with very fine sands, silty sands, and sandy silts with a thin covering of clay, as depicted by section D on plate 170. The thalweg position of course 13, as marked by a small slough immediately landward of the present seepage berm, was filled with approximately 40 ft of alternating strata of silty sands and sandy silts with occasional clay seams covered with 8 to 10 ft of clay.

464. The top stratum deposits landward of course 13 are typical of ridge and swale topography and consist largely of numerous shallow clay-filled swales with intervening sandy ridges (see plates 168 and 170). The site appears to be blanketed by natural levee deposits that cannot be distinguished with respect to grain size from the underlying point bar top stratum.

465. Relation of underseepage to geology. Except for heavy seepage during the 1945 high water, no serious sand boils have ever been reported for this site. However, prior to construction of the seepage berm in 1947, a potentially serious seepage condition dil exist between the toe of the levee and the deposits in the old slough landward of the levee (see section D on plate 170). The seepage berm now appears to adequatcly cover the thin top stratum previously existing along the levee toe.

466. Soil profiles and piezometer lines. Soil profiles and piezometer lines F, D, and B perpendicular to the levee are shown on plates 170 and 171. Soil profiles H and G in the borrow pits are shown on plate 172. Profiles I, K, and J, which are landward of and parallel to the levee, are shown on plates 173 and 174.

467. As shown on plate 168 and the soil profiles, the top stratum

beneath the levee consists of a relatively uniform layer of clay approximately 3 to 5 ft thick underlain by approximately 20 ft of alternating strata of silty sands and sandy silts (plate 173). Immediately landward of the present seepage berm, the top stratum consists of about 8 to 10 ft of clay in the bottom of an existing slough which is underlain by 20 to 30 ft of silts and silty sands. Landward of the slough, the topography varies as much as 10 ft in elevation with alternating shallow swales filled with clays and sandy ridges, both of which are underlain by 5 to 20 ft of silty sands and sandy silts (plate 170).

468. The tips of certain piezometers on line D were installed at a depth of approximately 50 ft and so as to cross the silt-filled channel previously described, for the purpose of determining if any significant drop in head occurs across this channel filling other than that which would normally be expected as a result of seepage flowing landward (see plate 170).

469. The pervious foundation at this site was explored at depth by only one boring (boring 49, plate 171). This boring indicates an upper stratum of fine to very fine sands approximately 40 ft thick underlain by approximately 60 ft of medium to coarse sands.

Analysis of piezometric
and seepage data

470. River stages and piezometer readings observed at Eutaw during the 1950 high water are plotted on plates 175 and 176. At the crest of this high water, H was about 9 ft above the natural ground along the toe of the existing seepage berm and the water in the slough immediately landward of the levee. The piezometric gradient existing in the pervious substratum beneath the levee along piezometer line D is shown on plate 177 for three selected river stages. Readings of piezometers along lines B and F are plotted on plate 178 for corresponding river stages. The substratum pressures along the toe of the existing levee and seepage berm are plotted on plate 179.

471. During the 1950 high water, the water level in piezometers with tips set in the silty sand immediately beneath the clay top stratum rose only about 0 to 1 ft above the natural ground surface. However,

pressure in the deeper underlying sand was as much as 6 ft above the elevation of water in the slough. Whether or not there was any excess head above the natural ground surface landward of the slough in the deep sands depended upon the ground elevation at the piezometer (see plate 177).

472. A summary of the information pertaining to the site and analysis of piezometric and seepage data subsequently discussed are given in table 24.

Table 24

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Eutaw, Miss., Site

Factor	Line D (Deep Sand) 1950 Flood	Line D (Deep Sand) Project Flood	Design Values Sta 2840-2880 Average	Design Values Sta 2840-2880 Slough	Upstream from Sta 2840 (Tentative)
River stage (crest)	141.2	163.1	163.1	163.1	163.1
Average el of ground or tailwater	135.0*	135.0*	135.0*	132.0	138.0**
Head on levee (H)	6.2	28.1	28.1	31.1	25.1
Piezometers used in analysis	D-2, -4, -7	-----	-----	-----	-----
Riverside borrow pit, width, ft	700	-----	700	-----	600
Top stratum	3 ft Cl- 5 ft Si Sd	-----	6 ft Si Sd	-----	6 ft Sd Si
Average stratum	6 ft Si Sd	-----	-----	-----	-----
Distance from riverside levee toe to river (L_1)	2500	-----	2500	-----	3500
Base width of levee (L_2)	450	-----	450	-----	250
Landward extent of top stratum (L_3)	-	-----	-	-----	150
Distance to effective seepage source (s)	1600	1500	1500	-----	1500
Effective length of riverside blanket (x_1)	1150	1050	1050	-----	-----
Distance to effective seepage exit (x_3)	1050	900	1000	-----	1000
Effective thickness of sand substratum (d)	70	-----	70	-----	70
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	1100	1100	1100	-----	1100
Laboratory permeability tests	560	-----	-----	-----	-----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	1310	-----	-----	-----	-----
Seepage and piezometric data	-----	-----	-----	-----	-----
Field pumping tests	-----	-----	-----	-----	-----
Well flow and piezometric data	-----	-----	-----	-----	-----
Top stratum, type	Sd Si	-----	Sd Si	Clay	Clay
Effective thickness for seepage analysis (z_{bL})	18	-----	18	-----	4.5
Critical thickness (z_t)	13	-----	-----	13	4.5
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	1.3	1.7	0.7	-----	-----
Piezometric data and blanket formulas	1.3	1.7	-----	-----	-----
Piezometric data and seepage measurements	-----	-----	-----	-----	-----
Permeability ratio (k_f/k_{bL})	875	645	800	-----	-----
Blanket formula	875	645	-----	-----	-----
Natural seepage measurements	-----	-----	-----	-----	-----
Natural seepage beneath levee					
Q_s, gpm/100 ft of levee	27	132	-----	-----	-----
Q_s/H, gpm/ft of head/100 ft of levee	4.3	4.7	-----	-----	-----

* Assumed average ground elevation landward of levee. Average ground elevation along toe of seepage berm = 132.0. Elevation of water in slough (1950 high water) = 132.0.

** Assumed tailwater with sublevee basin full.

473. Source of seepage. Seepage may enter the pervious foundation through the bank and channel of the Mississippi River approximately 2500 ft distant, in the bottom of the riverside borrow pits, and through the top stratum riverward of the levee.

474. Values of s at piezometer line D, for piezometer tips in deep sand, during the 1950 high water are plotted in fig. 39. The point of effective seepage entry into the underlying pervious foundation lies between the riverside borrow pit and the Mississippi River. At piezometer line D, s was about 1600 ft at the 1950 crest (fig. 39 and table 24). The fact that the source of seepage is this far from the levee can probably be attributed to the 5 to 10 ft of silts that blanket the riverside borrow pits. The distance to effective seepage entry is probably shorter for the Eutaw site in general than at piezometer line D because more clay seems to exist along D than at other sections perpendicular to the levee (see plates 168, 170, 171, and 172). Thus the primary source of seepage entry in the pervious foundation at the site appears to be in the riverside borrow pits. Also, it may be concluded that the average value of s for the site is probably somewhat less than that shown for line D in table 24.

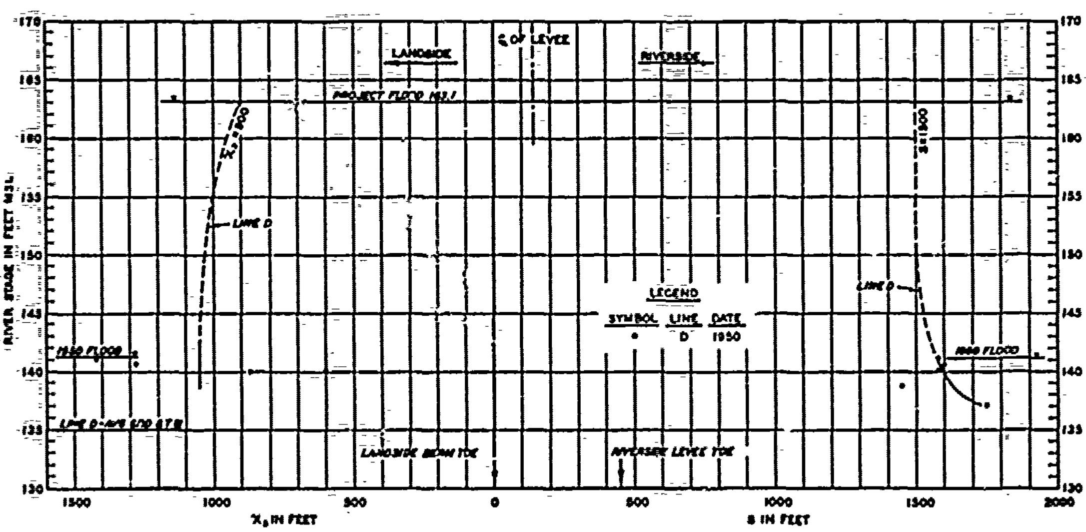

Fig. 39. Distances to effective seepage source and exit. Eutaw, line D (deep piezometers)

475. Seepage exit. Values of x_3 are plotted vs corresponding river stages in fig. 39 for the 1950 high water. The distance to the effective point of seepage exit was difficult to estimate because of the variation in tailwater and ground surface landward of the levee. In estimating x_3, it was assumed that the average ground and tailwater elevation was 135.0. The relation of this assumption to actual ground and tailwater elevations, as they existed at the crest of the 1950 high water, may be seen from plate 177. On the basis of the above assumption, the average x_3 at the crest of the 1950 high water was estimated to be about 1050 ft. On the basis of the maximum computed hydrostatic head that can exist landward of the levee at a project flood stage, it is believed that x_3 might decrease to about 900 ft for such a flood stage. The relatively long x_3 at Eutaw may be attributed to the relatively thick top stratum of sandy silts existing for a distance of approximately 900 ft landward of the present seepage berm.

476. Thickness and permeability of substratum sands. Relatively little information is available regarding the thickness and permeability of the sand aquifer. The pervious substratum is considered to have an effective thickness of about 70 ft. The gradations of typical foundation sands at the site are plotted on plate 166. The permeability of the pervious substratum was estimated from a correlation of D_{10} vs k_f, as shown in fig. 17, and results of a few laboratory tests on remolded samples. A $k_f = 1100 \times 10^{-4}$ cm per sec was estimated for the pervious substratum.

477. Thickness and permeability of top stratum. As previously described, the top stratum landward of the levee varies considerably in character and thickness (see plates 168, 170, and 171). The top stratum between the levee toe and the slough at piezometer A-1 consists only of about 3 ft of clay underlain by about 2 ft of sandy silt. The top stratum in the slough and landward thereof is believed to be similar to that shown for line D on plate 170. The top stratum from sta 2840 to 2880 landward of the present seepage berm consists essentially of a sandy silt-filled channel covered with about 8 ft of clay landward of which are alternating strata of clays and silts for a distance of about 600 ft

(see plates 168 and 170). In the seepage analyses, the top stratum was considered to be sandy silt 18 ft thick. The permeability of this top stratum, as computed from piezometric data obtained at the crest of the 1950 high water and effective thickness as shown in table 24, was 1.3 x 10^{-4} cm per sec.

478. Permeability ratio. The ratio of permeability of the foundation to that of the top stratum at piezometer line D is estimated to have been about 900 at the crest of the 1950 high water (see table 24).

479. Seepage flow. Seepage passing beneath the levee at line D at the crest of the 1950 high water, and for the project flood, was estimated using corresponding values of H, s, and x_3 for these floods (table 24). Natural seepage passing beneath the levee was estimated to be about 27 gpm per 100 ft of levee at the 1950 crest ($H = 6.2$ ft), or Q_s/H = about 4 gpm. At project flood stage, seepage passing beneath the levee may amount to as much as 130 gpm per 100 ft of levee, or Q_s/H = 5 gpm. On the basis of the hydraulic grade line shown on plate 177 for the 1950 high water, about 50 per cent of the seepage passing beneath the levee was emerging in an area about 1000 ft landward of the levee with the remainder either going into ground storage or emerging in a low area about 1500 ft from the levee.

480. Landside substratum pressures. Hydrostatic pressures that developed along the levee toe and seepage berm at the crest of the 1950 high water are shown on plate 179. Hydrostatic heads that developed landward of the seepage berm on line D as measured by piezometers immediately beneath the upper clay stratum and also in the deep pervious sands are shown on plate 177. Readings of selected piezometers along the levee toe vs river stages are plotted on plate 180. The head on the levee, top stratum characteristics, substratum pressures, and the gradient through the top stratum at certain typical piezometers are given in table 25.

481. The data on plate 176 indicate considerable lag in the development of pressure in shallow piezometers installed in the top of the silty sand and sandy silt stratum which lies immediately beneath the upper, more impervious clays. On the other hand, pressures in the

Table 25

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Eutaw, Miss., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (141.[illegible]): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (163.1): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
I	A-1[e]	138.0	-----	3.0	1.5	4.5	4.5	3.8	3.2	---	--	----	----	----	----	25.1	3.8[c]	15	9.0
	A-1[f]	130.0[g]	132.0	8.0	11.0	19.0	11.0	9.3	9.2	3.9	42	----	----	----	0.35	31.1[h]	9.3[c]	30	15.0
J	B-1	130.0[i]	132.0	11.5	0.0	11.5	11.5	9.8	9.2	1.0	11	----	----	----	0.09	31.1[h]	9.8[c]	32	----
J	C-1[f]	130.0[i]	132.0	9.0	2.5	11.5	9.0	7.7	9.2	1.3	14	----	----	----	0.14	31.1[h]	7.7[c]	25	----
D-J	D-6[f]	130.0[i]	132.0	8.0	0.0	8.0	8.0	6.8	9.2	0.3	3	----	----	----	0.04	31.1[h]	6.8[c]	22	15.3
D	D-7[j]	130.0[i]	132.0	8.0	2.0	28.0	13.0	11.0	9.2	6.0	65	----	----	----	0.46	31.1[h]	11.0[c]	35	12.3
D	D-8	140.0	-----	5.0	4.0	9.0	7.0	6.0	1.2	---	--	----	----	----	----	23.1[h]	6.0[c]	26	----
		130.0[k]	132.0	----	---	----	----	----	9.2	1.8	20	----	----	----	----	----	----	--	----
J	E-1[f]	128.0[i]	132.0	7.0	---	7.0	7.0	6.0	9.2	0.6	7	----	----	----	0.09	31.1[h]	6.0[c]	19	----
F-J	F-1[f]	128.0[i]	132.0	7.0	1.0	8.0	7.0	6.0	9.2	0.5	5	----	----	----	0.07	31.1[h]	6.0[c]	19	14.2

[c] See paragraph 143.
[e] In clean foundation sand.
[f] In channel fill silt.
[g] Bottom of slough 200 ft landward.
[h] Ground along toe of berm 132.0.
[i] Bottom of slough 30 ft landward.
[j] Deep piezometer in clean sand.
[k] Bottom of slough 100 ft riverward.

principal sand aquifer both at and landward of the levee toe responded rapidly to changes in river stage.

482. Uplift pressures that developed during the 1950 high water were not great enough to create any known sand boils landward of the levee. The fact that no sand boils occurred can be attributed largely to the low head of water against the levee and the relatively thick top stratum landward of the seepage berm. Based on the very limited data on plate 180 and in table 25, critical uplift pressures may develop landward of the levee at river stages higher than approximately 15 ft or a stage of some 16 ft below the project flood stage.

483. Excess heads along the toe of the present seepage berm at the crest of the 1950 high water ranged from only about 0 to 1 ft as measured by shallow piezometers (see plate 179). In view of the fact that the new seepage berm covers the thin top stratum between the original location of the landside levee toe and the slough, the most critical location as regards underseepage is in the slough and immediately landward thereof, except at piezometer A-1. It is along this slough that the greatest head can be expected to develop because of its low elevation and relatively low height to which water will probably impound. Because of the apparent lag in the shallow piezometers and the low head that developed during the

1950 high water, it is doubtful that the values of h_o/H shown in table 25 for shallow piezometers are as high as they may become during a sustained higher river stage. High pressures that may be expected to develop in the deep underlying sands are illustrated by the high ratio of h_o/H of 65 per cent at the crest of the 1950 high water (table 25). The prediction of the development of sand boils at river stages of about 15 ft is based on analyses of readings of both shallow and deep piezometers.

484. Sand boils may be expected at the north end of the seepage berm in the vicinity of piezometer A-1, because of the thinness of the top stratum between the landside toe of the levee and thicker top stratum deposits in the bottom of and landward of the slough paralleling the levee. Sand boils in this area may occur at a river stage of 147 or an H of only 9 ft. This is considered a critical location in regard to the development of excess pressures during high water. Conditions at piezometer A-1 are believed similar to those at the reach of levee along which the berm was constructed in 1947.

485. Gradients through the top stratum at the crest of the 1950 high water were quite low because of the low river stage.

Evaluation of seepage problem and
recommendations for control measures

486. An H of approximately 15 ft and a river stage of approximately 16 ft below project flood stage may create sand boils in the bottom of the slough paralleling the seepage berm. The same head may be expected to cause sand boils between the levee toe and the slough landward of the levee between sta 2835 to 2840, at the north end of the seepage berm (vicinity of piezometer A-1). On the basis of soil conditions and estimated excess hydrostatic pressures that can develop landward of the levee, the present seepage berm is believed too narrow for completely adequate seepage control and is thicker than required for its present width. Thus, some additional seepage control measures are recommended.

487. Irregularities in ground elevation and type and thickness of top stratum landward of the seepage berm make prediction of subsurface pressures at high river stages and the design of control measures difficult. The primary area of concern landward of the seepage berm is the

adjacent slough. In view of this, the most practical recommendation for additional seepage control measures appears to be the building of sublevees across the old slough at approximately sta 2820 and 2900, which would permit impounding of water up to el 138.0. These cross levees would have to be of substantial cross section and would have to be provided with manually operated control gates at each end with overflow spillways of adequate design set at el 138.0; the top of the cross levees should be set at el 140.0. The creation of such a sublevee basin would entail building a few low dikes along the landside bank of the slough. Impoundment of water to el 138.0 would reduce the maximum net head possible to 25 ft, and the estimated maximum excess head in the slough to 7 ft, which should be safe. Although the source of seepage at Eutaw is fairly far riverward of the levee, it is recommended that a few permeable dikes be constructed across the riverside borrow pits to encourage their filling, thus lengthening the distance to the effective source of seepage.

488. A potentially critical underseepage situation exists upstream of sta 2840 where a very thin and short reach of top stratum lies between the levee toe and much thicker landward top strata. Construction of the sublevee basin described above will not alleviate this situation. Additional seepage control measures recommended along this reach are either a relief well system or a landside berm.

L'Argent, Louisiana

489. L'Argent was selected as a site for study because a uniform, thick deposit of clay was known to exist over a broad area and the only source of seepage is in the river channel. The riverside borrow pits do not penetrate the thick clay top stratum. However, it was thought that high substratum pressures might develop because of the thickness and extensive area covered by the clay top stratum. Except for a sand boil 650 ft landward of the levee during the 1937 flood, no serious underseepage had been observed at this site.

Description of site

490. The site is located on the west bank of the Mississippi River

approximately 12 miles northeast of Ferriday, La., and extends from about levee sta 3525 to 3560. The levee is approximately 2800 ft from the river bank. Plans of the site, river, borrow pits, surface geology, topography, and piezometers are shown on plates 181 and 182; plate 183 is an aerial mosaic of the site.

491. At the site the levee crosses a former course of the Mississippi River which is now filled with a relatively thick deposit of clay. Although borrow pits 5 to 8 ft deep and 500 to 1000 ft wide have been excavated riverward of the levee, they do not penetrate to the underlying pervious foundation. The levee has a net height of approximately 30 ft. River stages at L'Argent can be estimated from the Natchez, Miss., gage and the graph on plate 191.

492. History of underseepage. During the 1937 high water when H = 20 to 24 ft, medium underseepage was reported between sta 3543 and 3546. A 6-in. sand boil occurred at sta 3528, 650 ft landward of the levee, and a 3-in. sand boil was reported at sta 3530, 250 ft landward of the levee.

493. During the 1945 high water, when H = 17.8 ft, no seepage was reported at the site. Correspondingly, no seepage or sand boils were reported at the site during the 1950 high water when H = 15.4 ft. The extent of the site is shown on plate 182; the locations of the 1937 sand boils are also plotted on this plate.

494. Piezometer installation. In 1948 one line of piezometers (B) was installed perpendicular to the levee at sta 3542+33 and another (line A) was installed along the landside toe of the levee from sta 3538 to 3552. The tips of most of the piezometers were set immediately beneath the clay stratum except for one, B-4, which was placed at about the middle of the top stratum as shown on plate 184. The first readings were made during the high water of 1950.

Geology of site and soil conditions

495. The general surface geology at the site is depicted on plates 181 and 182. Plate 181 shows the locations of former river courses, swales, and natural levee deposits which blanket a considerable portion of the area. The character and thickness of the top stratum in the

immediate vicinity of the site are shown in more detail on plate 182.

496. The central portion of the site is located on a relatively thick clay stratum deposited in a cutoff channel of course 14 (plates 181-185). The surface geology downstream of course 14 consists mainly of silty sands and sandy silts with frequent, rather shallow clay-filled swales having a trend approximately normal to the levee. This formation was created as point bar deposits during migration of the river from course 13 to course 14. A thin layer of natural levee silts and clays, difficult to distinguish from point bar and channel fillings with respect to grain size, is believed to cover most of the site. The clay filling in former river course 14 is relatively uniform along the landside toe of the levee, ranging in thickness from about 15 to 20 ft (see plate 185), but appears to thicken appreciably toward the present position of the Mississippi River (plate 184). The silty top stratum in the point bar deposits immediately downstream of course 14 is about 12 to 18 ft thick.

497. There is considerable relief in topography at the L'Argent site as shown by the contours on plate 182. It is to be noted that the ground surface at the toe of the levee is approximately 6 ft higher at sta 3550 than at sta 3535, and that the ground surface in the old channel filling of course 14 drops off appreciably landward of the levee.

498. Relation of underseepage to geology. The reason that no serious underseepage has occurred at L'Argent is probably due to the facts that no significant amount of seepage can enter the pervious foundation except along the bank of the Mississippi River 2500 to 3000 ft from the levee, and that the top stratum is thick enough to withstand uplift pressures that have occurred. The two sand boils noted during the 1937 high water occurred in areas where H was approximately 25 ft. On the basis of extrapolation of data obtained during the 1950 high water, the excess head in these areas was just about equal to H_c. The minor seepage reported between sta 3543 and 3546 may possibly have seeped through natural levee deposits that lie immediately beneath the base of the levee.

499. A potentially critical area at the site with respect to underseepage is believed to exist along the boundary of former river courses 13 and 14 (see plates 182-185). The probable reason that sand boils have

not occurred along this boundary is that the silty sand and sandy silt top stratum covering former course 13 is sufficiently pervious that natural leakage through the blanket reduces the substratum head enough to preclude serious seepage or sand boils. Another feature that may have significantly minimized seepage, both along the clay-filled channels and downstream point bar deposits, is the relatively low carrying capacity of the pervious substratum as subsequently discussed.

500. Soil profiles and piezometer lines. A generalized soil section perpendicular to the levee at the center of the site drawn to the same horizontal and vertical scales is shown at the top of plate 184. Detailed soil sections, profiles, and piezometer lines, both perpendicular and parallel to the levee toe, are shown on plates 184 and 185.

501. The full depth of the pervious aquifer was not penetrated by the borings at this site; however, from geological information the elevation of the top of Tertiary was estimated to be at -80 msl. If this estimate is correct, the pervious substratum at the site is approximately 120 ft thick. The upper 60 ft of the pervious sands are fine; no information is available regarding the lower half of the sand aquifer. The seepage carrying capacity of the aquifer at L'Argent is only about one-third of that for the piezometer sites previously discussed.

502. The clayey top stratum along piezometer line B at the center of the site is approximately 17.5 ft thick and thins to 14 ft at a point approximately 1500 ft landward from the center line of the levee, where point bar deposits of course 13 are encountered and the ground surface is approximately 10 ft higher than that in the channel filling (see plate 184). The average thickness of the clayey top stratum along the landside toe of the levee across the channel filling is very uniform, varying from about 17 to 19 ft; however, it is pointed out that the ground surface at sta 3530 is about 6 ft lower than at sta 3550 and thus the head on the levee at sta 3530 is always higher by a corresponding amount. The ground surface in the point bar deposits is the highest in the area; the thickness of the silty top stratum in this area ranges from about 12 to 18 ft (see plates 184 and 185).

503. Piezometer line B was located in a position thought to be

representative of the center of a wide clay-filled channel. Piezometers A-1 and C-1 were located along the landside toe of the levee for check purposes. Piezometers D-1 and D-2 were located at the boundary between courses 13 and 14 to observe substratum pressures at a point that was thought to be critical with respect to seepage.

Analysis of piezometric and seepage data

504. River stage and piezometer readings observed at the L'Argent site during the 1950 high water are plotted on plates 186 and 187. At the crest of the 1950 high water, H was about 16 ft. Piezometric gradients existing in the pervious substratum beneath the levee at piezometer line B are shown on plate 188 for selected river stages occurring during the 1950 high water. The hydrostatic head along the toe of the levee as measured by piezometers along line A is shown on plate 189. During the 1950 high water, h_o varied from about 1.5 to 5.5 ft along the toe of the levee. The highest heads were recorded at about sta 3538 where the ground is somewhat lower than in adjacent areas (plate 189). At piezometer line B the excess head was about 5 ft from the landside toe of the levee to the edge of the point bar deposits (course 13) about 1340 ft landward of the levee toe. A summary of information pertaining to the site and results of the piezometric and seepage data subsequently discussed are given in table 26.

505. Source of seepage. Values of s at piezometer line B during the 1950 high water are plotted in fig. 40. These values indicate that the seepage enters the sand substratum in the bed of the Mississippi River 2800 ft distant, and that the 10 to 15 ft of clay blanket in the riverside borrow pits effectively seals the pits. The distance to the effective source of seepage was about 3150 ft at the crest of the 1950 high water; it is estimated that s may be approximately 3000 ft at project flood stage. The source of seepage is comparatively distant from the levee at this site.

506. Seepage exit. The average ground surface and tailwater used in determining x_3 was taken to be at el 59, which represents an average of the tailwater elevations 800 ft landward as measured on 8 March

Table 26

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

L'Argent, La., Site

Factor	Line B 1950 Flood	Line B Project Flood	Design Values Sta 3526 to 3552
River stage (crest)	75.4	90.0	90.0
Average el of ground or tailwater	59.0	59.0	59.0
Head on levee (H)	16.4	31.0	31.0
Piezometers used in analysis	B-1 & B-2*	----	----
Riverside borrow pit, width, ft	1000	----	1000
Top stratum	12-20 ft clay	----	----
Average stratum	15 ft clay	----	15 ft clay
Distance from riverside levee toe to river (L_1)	2500	----	2500
Base width of levee (L_2)	380	----	380
Landward extent of top stratum (L_3)	1340	----	1340
Distance to effective seepage source (s)	3150	3000	3000
Effective length of riverside blanket (x_1)	2770	2620	2620
Distance to effective seepage exit (x_3)	3000	4200	5500
Effective thickness of sand substratum (d)	120	----	120
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	400	----	400
Laboratory permeability tests	----	----	----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	350	----	----
Seepage and piezometric data	----	----	----
Field pumping tests	----	----	----
Well flow and piezometric data	----	----	----
Top stratum, type	Clay	----	Clay
Effective thickness for seepage analysis (z_{bL})	15.5	----	15.5
Critical thickness (z_t)	15.5	----	15.5
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	0.24	0.15	----
Piezometric data and blanket formulas	0.24	0.15	----
Piezometric data and seepage measurements	----	----	----
Permeability ratio (k_f/k_{bL})	1700	2700	3700
Blanket formula	1700	2700	----
Natural seepage measurements	----	----	----
Natural seepage beneath levee			
Q_s, gpm/100 ft of levee	18	30	----
Q_s/H, gpm/ft of head/100 ft of levee	1.1	1.0	----

* Head at B-2 was increased to obtain average head in sand substratum as described in paragraph 132.

(plate 188) and the ground surface at the landside toe of the levee. From the data plotted in fig. 40, x_3 appears to increase progressively during rising river stages up to the crest of the 1950 flood. The distance to the effective seepage exit at project flood stage based on the maximum h_o that can exist at line B is about 4200 ft. It is believed that x_3 will increase with rising river stages until the river reaches el 80 to 85, and will then decrease as the river rises above el 85 (see

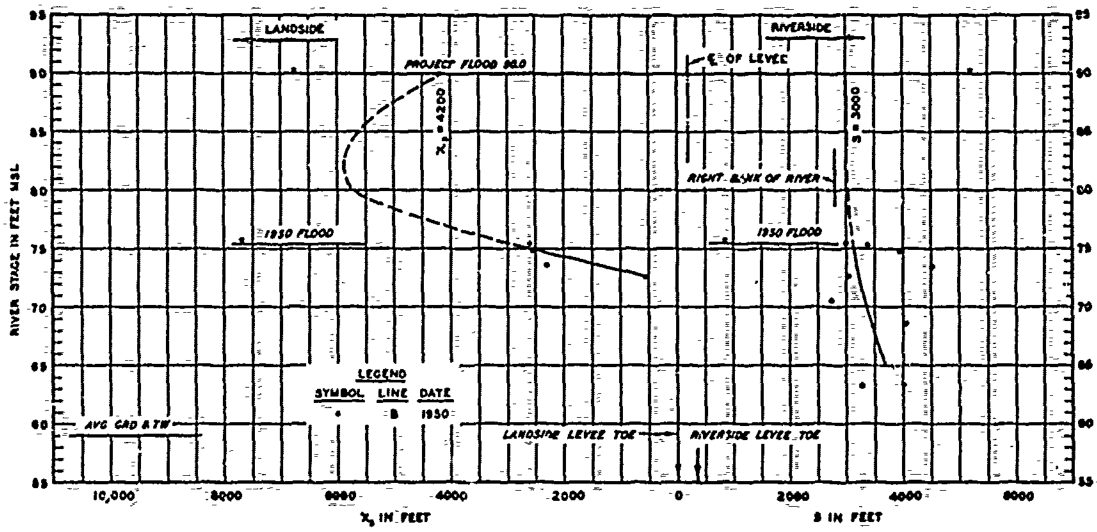

Fig. 40. Distances to effective seepage source and exit. L'Argent, line B

fig. 40). The above phenomenon of an increasing x_3 followed by a decrease is similar to that at line E, Trotters 51.

507. Thickness and permeability of the substratum sands. The pervious foundation at L'Argent consists of medium fine, and fine to medium sands to a depth of about el -35. The top of Tertiary is estimated at about el -80, and as borings did not penetrate below -35, about 45 ft of aquifer were not explored. From grain-size data and fig. 17 the permeability of the sands above el -35 was estimated to be about 200×10^{-4} cm per sec. The average permeability of the sands below el -35 was assumed equal to that at the nearby Hole-in-the-Wall site (500×10^{-4} cm per sec). On the basis of these values and assuming an effective thickness of the sand substratum of 120 ft, it is estimated that $k_f = 400 \times 10^{-4}$ cm per sec at L'Argent.

508. Thickness and permeability of top stratum. The top stratum landward of the levee (line B) consists of about 15.5 ft of clay for a distance of about 1340 ft. Landward of this point the top stratum consists of about 17 ft of silt and silty sand with clay strata. The permeability of the clay top stratum computed from piezometric data obtained during the crest of the 1950 high water was found to be about 0.2×10^{-4} cm per sec. Estimated values of k_{bL} at the project flood are given in table 26.

509. Permeability ratio. The ratio of permeability of the foundation to that of the top stratum at the crest of the 1950 high water is estimated to have been 1700; values for project flood stage are given in table 26.

510. Seepage flow. The natural seepage passing beneath the levee at the crest of the 1950 high water was estimated to be 18 gpm per 100 ft of levee, or Q_s/H = about 1.1 gpm. It is estimated that at the project flood Q_s will be about 30 gpm per 100 ft of levee, or Q_s/H = about 1.0 gpm (table 26). The lower value of Q_s/H for the project flood as compared with that at the 1950 crest is predicated on the longer x_3 at the project flood. On the basis of these seepage estimates, the site is subject to a low rate of natural seepage.

511. Landside substratum pressures. The hydrostatic pressures that developed along the toe of the levee at or near the crest of the 1950 high water are shown on plate 189, line A. Readings of selected piezometers vs river stage at the landside toe of the levee are plotted on plate 190. Also shown are estimated substratum pressures and maximum estimated piezometer readings for river stages up to the project flood. The head on the levee, top stratum characteristics, and substratum pressures at certain typical piezometers along the landside toe of the levee are given in table 27. From plates 186 and 187 it may be noted that the maximum heads landward of the levee lagged about a week behind the maximum river stage observed in 1950. There was also about a two weeks lag

Table 27

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

L'Argent, La., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (75.4): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (90.0): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
A	A-1	59.0	----	20.0	0.0	20.0	20.0	17.0	16.4	5.6	34	----	----	----	0.28	31.0	----	--	----
B-A	B-2	60.0	----	17.5	0.0	17.5	17.5	14.9	15.4	5.4	35	----	----	----	0.31	30.0	14.9[c]	50	22.6
B	B-3	56.2	57.5	14.5	0.0	14.5	14.5	12.3	17.9	5.1[e]	29	----	----	----	0.35	32.5	12.3[c]	38	23.0
A	C-1	62.5	----	16.0	2.0[f]	18.0	18.0	15.3	12.9	3.1	24	----	----	----	0.17	27.5	15.3[c]	56	20.7
A	D-1	64.5	----	4.2	4.0	8.2	7.2	6.1	10.9	0.4	4	----	----	----	0.06	25.5	0.6[d]	2	9.7
A	D-2	64.5	----	4.2	7.8	12.0	10.1	8.6	10.9	1.4	13	----	----	----	0.14	25.5	8.6[c]	34	15.7

c, d See paragraph 143.

e h_x at B-3.

f Silt on top of clay.

in the development of substratum pressures above the ground surface after the river reached a bankfull stage. This lag is attributed to the filling of natural ground storage landward of the levee as the river rose and the relatively low permeability of the pervious aquifer.

512. Sand boils did not develop at L'Argent during 1950. Uplift pressures along the toe of the levee varied from about 5.6 ft at piezometer A-1 to 1.4 ft at piezometer D-2 at the crest of the 1950 high water, and correspond to values of 34 and 13% H, respectively. (It should be noted that the ground surface is about 4.5 ft higher at piezometer D-2 than at A-1.) The maximum upward gradients through the top stratum were about 0.30 to 0.35. Uplift pressures as high as 5.1 ft were recorded at piezometer B-3, about 1200 ft landward of the levee.

513. On the basis that $i_c = 0.85$, the clay top stratum between about sta 3526 and 3552 can withstand about 12 to 17 ft of excess head. Similarly, the silty top stratum between sta 3552 and 3557 can withstand about 9 ft of excess head. From the data given on plate 190 and in table 27, it is estimated that critical substratum pressures will develop between sta 3526 and 3557 at a river stage of about 22 ft, or 9 ft below the project flood stage.

Evaluation of seepage problem and recommendations for control measures

514. The levee at L'Argent has no landside seepage berm. As indicated above, the existing top stratum between about sta 3526 and 3557 probably is not sufficiently thick to prevent the formation of critical uplift pressures landward of the levee toe at river stages higher than about 20 ft. Thus, some seepage control measures are indicated. The riverside borrow pits are already sufficiently blanketed with clay and the effective seepage source is in the river. In view of the thickness and low permeability of the top stratum, and because of the discontinuity existing in the top stratum landward of the levee, a line of relief wells would probably be the most practical control measure at this site. The wells not only would reduce the pressure at the levee toe but also landward of the levee, and would prevent the formation of sand boils and possible piping. Some seepage control measures may also be required both

upstream and downstream from the reach of levee between sta 3526 and 3557. Sufficient soils and piezometric data are not available, however, to make a decision in this regard nor to design the control measures, if required.

Hole-in-the-Wall, Louisiana

515. Hole-in-the-Wall was selected as a site for study because it is representative of relatively uniform thin deposits of clay and sandy silts over a relatively broad area; heavy underseepage and sand boils had been observed there during the 1937 high water; and a wide seepage berm has since been constructed but not tested by a high river stage.

Description of site

516. The site, located immediately downstream of the L'Argent site, extends from levee sta 3600 to 3630. A general plan of the area is shown on plate 181; more detailed plans of the site, river, borrow pits, surface geology, topography, and piezometers are shown on plate 192; plate 183 is an aerial mosaic of the site. The levee is approximately 2200 ft from the Mississippi River and has a net height of approximately 23.5 ft. Riverside borrow pits 5 to 10 ft deep and 500 to 1000 ft wide extend along the site, and in some locations have penetrated the underlying sand stratum but, in general, the pits are blanketed by 5 to 8 ft of silts and clay silts. River stages at the site can be estimated from the Natchez, Miss., gage and the graph on plate 191.

517. History of underseepage. During the 1937 high water, when $H =$ about 14.7 ft, heavy underseepage was reported between sta 3597 and 3637; numerous small sand boils were reported between sta 3610 and 3625. A large seepage berm approximately 6 ft thick at the levee toe and 200 ft wide was constructed in 1940. Sections of the berm are shown on plates 193-195. During a high water in 1945 ($H = 11.4$ ft), medium underseepage was reported from the levee toe to a distance 500 ft landward between sta 3597 and 3625. Light underseepage was observed landward of the levee between sta 3597 and 3625 during the 1950 high water ($H = 9.4$ ft), but no sand boils were reported.

518. Piezometer installation. In 1948 four lines of piezometers

(B, C, D, and E) were placed perpendicular to the levee. Some of the piezometers on lines B and E were installed riverward of the levee. Additional piezometers (line A) were installed along the landside toe of the seepage berm. Piezometer readings of consequence were first obtained during the 1950 high water.

Geology of site and soil conditions

519. The general geology of the site is illustrated on plate 181; the type and thickness of top stratum deposits are shown in more detail on plate 192. The site is located on point bar deposits laid down when the river shifted its channel from course 11 to course 14 (see plate 181). The site as discussed in this report is located in former river course 12. Numerous shallow, clay-filled swales trending approximately normal to the levee extend under the levee and for some distance landward. An exceptionally broad and deep clay-filled swale roughly parallels the levee on the riverside at a distance of about 1700 to 1800 ft (plates 192 and 193). In general, the surface geology landward of the levee for 1000 ft consists of a thin top stratum of clays and clay silts 2 to 5 ft thick, underlain by sandy silts and silty sands 2 to 5 ft thick. The top stratum is rather variable as regards both character of soil and thickness. The upper clay stratum riverward of the levee has been completely removed as a result of borrow operations. Silty and clayey natural levee deposits cover much of the site, and it is possible that most, if not all of the fine-grained top stratum is natural levee deposits. Owing to the difficulty of separating natural levee deposits from the underlying fine-grained top stratum of point bar material, no distinction between the two has been made on the soil profiles.

520. Relation of underseepage to geology. Geologically, the top stratum deposits are essentially of the same character and vary only as is normal for point bar deposits. The only relation between geology and location of sand boils reported during the 1937 high water between sta 3610 and 3625 is that their location coincides quite well with that reach of top stratum along the levee toe where the top stratum is thinnest (see plate 192). Although the clay-filled swale riverward of the levee is quite wide and probably as deep as 40 ft, it is not considered to have

any significant effect on seepage flow, in that seepage may enter the pervious foundation at the riverbank and through riverside borrow pits.

521. Soil profiles and piezometer lines. The locations of piezometers and borings are shown on plates 181 and 192. Soil profiles and piezometer lines both perpendicular and parallel to the landside toe of the levee are shown on plates 193-195. Because of inability to always determine the bottom of the top stratum at this site, the tips of a number of piezometers were set at two different elevations at the same location to determine the hydrostatic pressure more precisely beneath or in the top stratum.

522. The sediments making up the top stratum in the point bar area vary from lean clay to silty sand. In general, they consist of 1 to 5 ft of lean clay or clay silt underlain by 2 to 5 ft of silty sands and sandy silts. There probably are some slightly deeper clay swales in the area which were not encountered by the boring explorations.

523. The pervious substratum consists of alternating strata of fine to medium sands and has an estimated thickness of approximately 140 ft. The effective permeability of the sand stratum at this site is estimated to be about 500×10^{-4} cm per sec, or only 1/3 to 1/2 of that at most piezometer installations in the Memphis and Vicksburg Districts. The seepage-carrying capacity of the aquifer is low which is probably the reason that underseepage at this site was not more serious during the high waters occurring to date.

Analysis of piezometric and seepage data

524. River stages and piezometer readings observed during the 1950 high water are plotted on plates 196 and 197. At the crest of this high water H was about 10 ft. Piezometric gradients existing in the pervious substratum beneath the levee along piezometer lines B, D, and E, perpendicular to the levee, are shown for selected river stages on plates 198 and 199. The hydrostatic head along the toe of the levee as measured by piezometers along line A is also plotted on plate 199, and at the 1950 crest ranged from about 0.5 to 2.0 ft above the average elevation of the ground surface. Little difference in head existed between piezometers

immediately beneath the top stratum and those about 4 to 5 ft deeper along the toe of the levee, as shown on plate 199 (line A). Excess heads above the ground surface were observed only a short distance landward of the levee, as shown on plates 198 and 199. A summary of information pertaining to the site and the results of analysis of piezometric and seepage data subsequently discussed are given in table 28.

525. Source of seepage. Seepage may enter the pervious aquifer through the bank and bed of the Mississippi River, approximately 2200 ft distant, and also through riverside borrow pits excavated along the levee.

526. Values of s at piezometer lines B, D, and E during the 1950 high water are plotted in fig. 41. The values of s shown in fig. 41 and plates 198 and 199 indicate that seepage enters the sand substratum primarily through both riverside borrow pits and the natural top stratum riverward of the levee. A portion of the natural blanket has been removed in the borrow pits at lines B and D (plates 194-195) and almost all

Table 28

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Hole-in-the-Wall, La., Site

Factor	Line B 1950 Flood	Line B Project Flood	Line D 1950 Flood	Line D Project Flood	Line E 1950 Flood	Line E Project Flood	Design Values Sta 3603-3606	Design Values Sta 3606-3623	Design Values Sta 3623-3632
River stage (crest)	75.4	89.5	75.4	89.5	75.4	89.5	89.5	89.5	89.5
Average el of ground or tailwater	66.0	66.0	65.0	65.0	66.0	66.0	65.0	66.0	66.0
Head on levee (H)	9.4	23.5	10.4	24.5	9.4	23.5	24.5	23.5	23.5
Piezometers used in analysis	B-2, B-4	----	D-2, D-3, D-5	----	E-2, E-3, E-5	----	----	----	----
Riverside borrow pit, width, ft	500	----	700	----	800	----	500	700	800
Top stratum	2-3 ft Cl Si 2 ft Sd Si	----	----	----	0-3 ft Si Sd	----	----	----	----
Average stratum	3 ft Cl Si	----	3 ft Si Sd	----	1 ft Si Sd	----	3 ft Cl Si	2 ft Si Sd	2 ft Si Sd
Distance from riverside levee toe to river (L_1)	2200	----	2100	----	2100	----	2200	2100	2100
Base width of levee (L_2)	500	----	500	----	500	----	500	500	500
Landward extent of top stratum (L_3)	∞	----	∞	----	∞	----	∞	∞	∞
Distance to effective seepage source (s)	1150	1150	2100	2100	1850	1850	1200	1200	1200
Effective length of riverside blanket (x_1)	650	650	1600	1600	1350	1350	700	700	700
Distance to effective seepage exit (x_3)	----*	120	600	300	290	700	625	500	535
Effective thickness of sand substratum (d)	130	----	130	----	130	----	130	130	130
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	500	----	500	----	500	----	500	500	500
Laboratory permeability tests	----	----	60	----	----	----	----	----	----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	----	----	500	----	----	----	----	----	----
Seepage and piezometric data	----	----	----	----	----	----	----	----	----
Field pumping tests	----	----	----	----	----	----	----	----	----
Well flow and piezometric data	----	----	----	----	----	----	----	----	----
Top stratum, type	Cl & Sd Si	----	Cl & Sd Si	----	Cl & Sd Si	----	Cl Si	Cl & Sd Si	Cl
Effective thickness for seepage analysis (z_{bL})	8.0	----	4.0	----	8.0	----	7.5	4.0	5.5
Critical thickness (z_t)	5.0	----	2.2	----	7.5	----	7.5	4.0	5.5
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	----	36	0.72	2.9	6.2	1.1	----	----	----
Piezometric data and blanket formulas	----	36	0.72	2.9	6.2	1.1	----	----	----
Piezometric data and seepage measurements	----	----	----	----	----	----	----	----	----
Permeability ratio (k_f/k_{bL})	----	14	690	173	81	470	400	400	400
Blanket formula	----	14	690	173	81	470	----	----	----
Natural seepage measurements	----	----	----	----	----	----	----	----	----
Natural seepage beneath levee (computed)									
Q_s, gpm/100 ft of levee	----	----	40	98	42	88	----	----	----
Q_s/H, gpm/ft of head/100 ft of levee	----	----	3.5	4.0	4.5	3.7	----	----	----

* It was not possible to determine x_3 in 1950, as the landward extension of the hydraulic grade line intersected the ground surface beneath the levee or berm, for all river stages experienced.

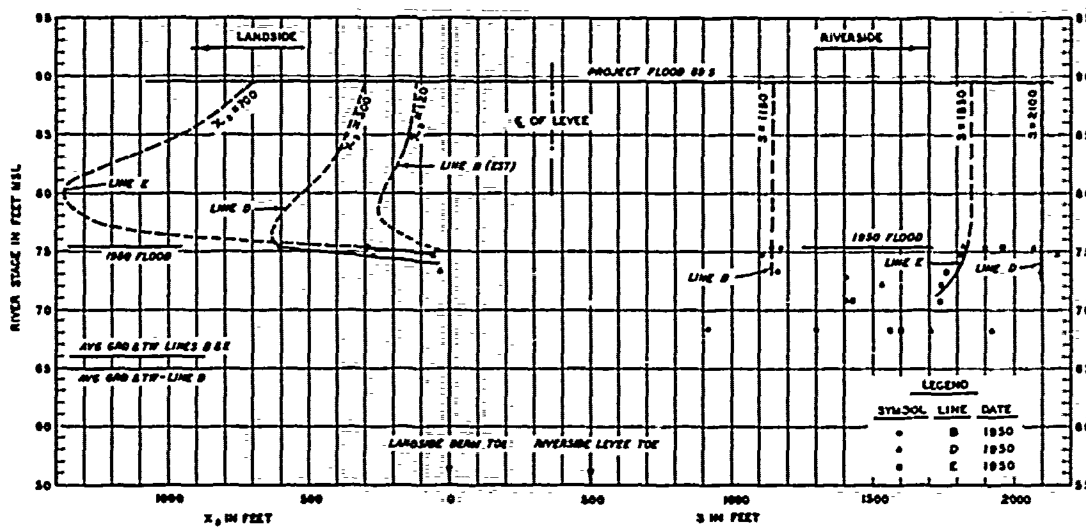

Fig. 41. Distances to effective seepage source and exit. Hole-in-the-Wall, lines B, D, and E

of the top stratum has been removed in the bottom of the borrow pits riverside of the levee at line E, plate 193. In general, some thin strata of silty sands and sandy silts remain in the bottoms of the borrow pits along the site. At piezometer line B, s was about 1150 ft at the crest of the 1950 high water. As excess heads first developed landward of the levee at a river stage of about 8 ft, a state of truly artesian flow did not exist at lower river stages. As a result, values of s determined for river stages less than 8 ft may not be reliable. At lines D and E, s was about 2100 and 1850 ft, respectively, at the crest of the 1950 high water. Because values of s that are considered reliable were obtained only at about the 1950 crest, it was difficult to determine any trend in s with river stage. Therefore, it was assumed that at the project flood s would be the same as at the 1950 crest. It should be noted that values of s are comparatively large considering the fact that the top stratum remaining in the bottom of the borrow pits is probably relatively pervious.

527. Seepage exit. Values of x_3 are plotted vs corresponding river stages in fig. 41 for the 1950 high water. As explained above, since artesian flow conditions did not develop beneath the levee until near the crest of the flood, values of x_3 obtained at river stages less

than el 74 are meaningless. After excess heads developed above the ground surface, the apparent values of x_3 increased considerably as the river rose. At lines D and E, x_3 was about 600 and 300 ft, respectively, at the flood crest. At piezometer line B, the landward extension of the portion of the gradient line beneath the levee intersected the ground surface riverward of the landside berm toe during the entire 1950 high water and as a result all values of x_3 are meaningless. The relationship between x_3 and river stage as shown in fig. 41 for stages in excess of the 1950 flood was computed from equation 31 and piezometer readings plotted against river stage extrapolated to the project flood (for example, see plate 200). It should be noted that at line B, all values of x_3 shown in fig. 41 were estimated, as measured values of x_3 during the 1950 high water were negative. Although x_3 at lines E and B should be about the same, inasmuch as the landside top stratum at both lines consists of about 8 ft of clay and sandy silt, the estimated values of x_3 at line B were considerably less. On the basis of the maximum computed hydrostatic head that can exist landward of the levee at the project flood stage (assuming $i_c = 0.85$), x_3 is estimated at about 100 to 700 ft for the project flood stage. As seen from fig. 41, x_3 will probably increase until the river rises to about el 80 ($H \cong 15$ ft), at which time seepage and sand boils landward of the levee will probably be sufficient to cause a reduction in x_3 for subsequent rising river stages.

528. Thickness and permeability of substratum sands. The pervious substratum at Hole-in-the-Wall consists of a deep stratum of pervious sands, varying in gradation from fine to medium, and having an effective thickness of about 130 ft. The gradations of typical foundation sands at the site are plotted on plate 191. The permeability of the pervious substratum was estimated from grain-size data to be 500×10^{-4} cm per sec, or only one-third to one-half of that at most piezometer installations upstream of this area. Values of k_f as estimated from laboratory permeability tests and grain size data are given in table 28.

529. Thickness and permeability of top stratum. The top stratum landward of the levee consists of a surface layer of clay underlain by

sandy silts and silty sands, the total thickness of which varies between about 5 and 8 ft. The effective thickness of the top stratum used in the seepage analyses made at piezometer lines B, D, and E is given in table 28. The estimated permeability of the combined top stratum thickness (clays and silts) at the crest of the 1950 high water varied between 0.7 and 6×10^{-4} cm per sec.

530. Permeability ratio. The ratio of permeability of the foundation to that of the top stratum was estimated at about 700 for piezometer line D and only 80 at line E at the 1950 high-water crest. Estimates of the permeability ratio for the project flood at lines B, D, and E are given in table 28.

531. Seepage flow. Seepage passing beneath the levee at the crest of the 1950 high water, and for the project flood, was estimated using the corresponding values of H, s, and x_3 for these floods (see table 28). At the 1950 crest, the natural seepage passing beneath the levee was computed to be about 40 gpm per 100 ft of levee or $Q_s/H = 4$ gpm. At project flood stage, it is estimated that Q_s may amount to as much as 90 to 180 gpm per 100 ft of levee (Q_s/H = about 4 to 8 gpm). On the basis of these data, natural seepage at Hole-in-the-Wall during high waters can be classed as medium.

532. Landside substratum pressures. Hydrostatic pressures that developed along the toe of the berm (line A) at the crest of the 1950 high water are shown on plate 199. Hydrostatic heads landward of the levee at the 1950 crest are shown on plates 198 and 199. Readings of selected piezometers at the toe of the levee vs river stages are plotted on plate 200. The head on the levee, top stratum characteristics, substratum pressures, and the upward gradient through the top stratum at certain typical piezometers are given in table 29.

533. The data on plates 196-197 show that considerable time was required for the ground-water storage landward of the levee to fill, and the piezometers landward of the levee did not record excess heads until about 30 days after the river reached bankfull stage. Excess heads landward of the levee did not begin to develop until H = about 8 ft. This considerable lag in the development of excess heads is attributed to slow

Table 29

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Hole-in-the-Wall, La., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft				h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (75.4)			Est Gradient through Top Stratum (1950 Flood)				Project Flood (89.5)			Est H at i_c ft
				Clay	Silt	Total	z_t		H ft	h_o ft	$\frac{h_o}{H}$ %	Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	H ft	h_o ft	$\frac{h_o}{H}$ %	
A	A-1	64.5	----	7.5	3.0	10.5	7.5	6.4	10.9	2.0	18	----	----	----	0.27	25.0	6.4[c]	26	----
B-A	B-5	66.0	----	2.5	0.0	2.5	2.5	2.1	9.4	0.5	5	----	----	----	0.20	23.5	2.1[c]	9	11.2
C-A	C-1	65.5	----	1.5	7.5	9.0	9.0	7.6	9.9	1.4	14	----	----	----	0.16	24.0	7.6[c]	32	----
D-A	D-4	66.0	----	1.2	1.0	2.2	2.2	1.9	9.4	1.2	13	----	----	0.54	----	23.5	1.9[c]	8	10.3
D	Avg D-6 D-7	63.0	64.0	2.0	3.0	5.0	5.0	4.3	11.4	2.0	18	----	----	0.40	----	25.5	4.3[c]	16	----
E-A	E-4	66.0	----	2.0	6.5	8.5	7.5	6.4	9.4	1.2	13	----	----	----	0.16	23.5	6.4[c]	27	12.5
A	F-1	66.0	----	5.5	0.0	5.5	5.5	4.7	9.4	0.7	7	----	----	----	0.13	23.5	4.7[c]	20	15.8

[c] See paragraph 143.

filling of the natural ground storage landward of the levee as the river rose because of the apparent relatively great distances to the source of seepage and the low permeability of the foundation compared to that of the other sites farther upstream.

534. Uplift pressures during the 1950 high water were not great enough to cause any known sand boils landward of the levee, and in general only light to medium underseepage occurred at the site. The fact that no sand boils occurred can probably be attributed to the low head of water against the levee (about 10 ft), the low k_f , and the relatively great distance to the source of seepage. However, based on plots on plate 200 and data in table 29, it appears that critical uplift pressures will develop landward of the levee at river stages higher than approximately 10 to 16 ft. The project flood stage will create an H of about 24 to 25 ft.

535. Excess heads landward of the levee ranged from about 0.5 to 2.0 ft at the crest of the 1950 high water, or about 5 to 18% H . It is believed that excess heads of about 2 to 6 ft will be required before sand boils will occur. Estimated gradients through the top stratum at the crest of the 1950 high water were comparatively low and varied between 0.13 and 0.54 (table 29).

Evaluation of seepage problem and recommendations for control measures

536. An H of about 10 ft during the 1950 high water was not

sufficient to develop sand boils at the site. However, the width of the existing berm is not believed adequate to prevent the development of critical substratum pressures and sand boils landward of the berm toe at net heads higher than about 12 to 15 ft. As H during project flood will be about 24 ft, either a moderate extension of the present berm or installation of relief wells to supplement it is recommended. The existing berm is considered to have adequate thickness (see plate 198).

Kelson, Louisiana

537. Kelson was selected for study and installation of piezometers because it was considered representative of sites where the levee is underlain by natural levee deposits of sandy silt in turn underlain by thick clay and then pervious sand. Heavy underseepage had been reported in 1937 at Kelson.

Description of site

538. The site is located on the west bank of the Mississippi River approximately 15 miles north of Baton Rouge, La. Plans of the site, river, borrow pits, surface geology, topography, and piezometer layout are shown on plates 201 and 202; plate 203 is an aerial mosaic of the site. The site, as discussed in this report, extends from levee sta 2672 to 2710. The levee is located approximately 800 ft from the bank of the Mississippi River; however, as shown on plate 202, a wide, flat sand bar lies between the top of bank and normal channel of the river. The levee has a net height of approximately 22 ft. Rather extensive borrow pits have been excavated riverward of the levee as shown on plates 202, 204, and 205, but these pits have not penetrated the deep underlying sands. River stages at the site can be estimated from the Baton Rouge gage and the graph on plate 211.

539. History of underseepage. During the 1937 high water, when H = 18 ft, heavy underseepage was reported along the toe of the levee and in fields landward between sta 2680 and 2750. No seepage was observed during the 1950 high water, which created a sustained head of almost 17 ft on the levee for several weeks. In view of the lack of

seepage in 1950 and soil conditions subsequently discussed, the report of heavy underseepage during the 1937 high water is questionable.

540. Piezometer installation. In 1948 one line of piezometers was installed perpendicular to the levee at sta 2700+71 (line B). In general the tips of the piezometers were installed in a silty stratum beneath a surface clay stratum. The first readings were obtained during the 1950 high-water period.

Geology of site and soil conditions

541. The general geology of the site is illustrated on plate 201; the type and thickness of the top stratum are shown in more detail on plate 202. The site is located mainly on channel fill and natural levee deposits. The channel filling of clay underlying the site was deposited in former river course 13 after a chute cutoff occurred in this stage some distance upstream. The clayey filling in this old channel varies from 10 to 15 ft in thickness (plate 204). Subsequent to the filling of river course 16 the area was covered by natural levee deposits composed largely of clayey silts and sandy silts to a depth of about 10 ft. The levee is founded directly on these natural levee deposits.

542. Relation of underseepage to geology. At the time the Kelson site was selected for investigation there was no reason to doubt the heavy underseepage reported during the 1937 high water. Accordingly a detailed investigation was begun to learn whether the cause of the reported heavy underseepage was the natural levee deposits immediately under the levee or the result of seepage and pressure in the deep underlying pervious foundation.

543. It now appears that any underseepage occurring at this site in 1937 can be related only to seepage through silt seams in the lower part of the natural levee deposits underlying the levee. Seepage probably gained access to these silt seams where they had been exposed in the sides of the riverside borrow pits. Appreciable seepage from the underlying sand substratum appears to be precluded by the presence of the thick silt and clay beds that underlie the site, and the fact that no seepage was observed during the 1950 high water. The combined silt and clay strata seem to be sufficiently thick to withstand any substratum

pressures that might develop in the lower sands, even at project flood stage. The clay and silt deposits filling river course 13 have no influence on the development of seepage through the natural levee deposits under the levee or from the deeper underlying sands, since the natural levee deposits laid down during courses 19 and 20 extend without interruption across both courses 9 and 13, and the upper clay strata in course 13 are no thicker than, if as thick as, the clay top stratum between the levee and course 13.

544. Soil profiles and piezometer lines. The locations of piezometers and borings are shown on plates 202 and 203. Soil profiles and piezometer lines both perpendicular and parallel to the landside toe of the levee are shown on plates 204-206. The tips of the piezometers, except for piezometer A-5, are located within the natural levee deposits underlying the levee. The tip of piezometer A-5 is located beneath the previously discussed clay-channel filling that covers the area and is at such elevation that it is believed to reflect the hydrostatic head in the underlying pervious sand formation fairly accurately.

545. The natural levee deposits are quite heterogeneous in character and thickness. At piezometer line B, they consist essentially of sandy silts approximately 5 to 8 ft thick extending from the riverside toe of the levee to 100 to 200 ft landward. At line B the sandy silts are overlain by 1 to 2 ft of more impervious silty clay. Variations in the type and thickness of the natural levee deposits are well illustrated by the logs of borings shown on plates 204 and 205.

546. Relatively little exploration was made of the pervious sand formation underlying the site. From other geological information, the thickness of the sand was estimated to be approximately 90 ft. The upper 40 to 50 ft of this sand stratum consists of very fine sand. Laboratory tests on remolded samples indicate a permeability of about 2×10^{-4} cm per sec (see boring 2-X, plate 204); however, these sands probably have a considerably higher k_f in situ. No information is available on the permeability of the lower sands.

Analysis of piezometric and seepage data

547. River stages and piezometer readings observed at Kelson during the 1950 high water are plotted on plates 207 and 208. This high water created a maximum H of about 17 ft, or only 5 ft below project flood stage. Piezometric gradients in the silt stratum beneath the levee are shown for selected river stages on plate 209.

548. No hydrostatic head was observed in the silt stratum above ground surface during the 1950 high water. However, an excess head of 4.6 ft did exist beneath the thick clay stratum underlying the levee.

549. A summary of information pertaining to the site and the results of analyses of seepage and piezometric data subsequently discussed are given in table 30.

550. Source of seepage. The only known source of seepage into the deep underlying sands is in the channel of the existing river or through the sand bar shown on plates 202 and 204. Seepage can enter the sandy silt stratum of the lower part of the natural levee deposit through the levee side of the riverside borrow pits (see plates 204 and 205).

551. The distance to the effective source of seepage entry into the silt stratum beneath the levee during the 1950 high water is plotted in fig. 42. The data plotted on plate 209 and in fig. 42 show that at line B the seepage enters the silt stratum in the bottom of the borrow pit adjacent to the levee, s being only 250 ft from the landside toe of the levee. The value of s decreased somewhat as the river rose. This possibly can be attributed to scouring away of impervious materials which may have accumulated over the exposed face of the silt stratum in the riverside borrow pit. It was not possible to estimate the distance to the effective source of seepage entry into the deep sand formation from the one piezometer set in this stratum.

552. Seepage exit. Values of x_3 for the silt stratum beneath the levee are plotted vs corresponding river stages in fig. 42 for the 1950 high water. The average x_3 at the crest of the 1950 high water was about 50 ft. The very short distance to the seepage exit is attributed to the practically complete lack of top stratum over the silty stratum landward of the levee.

Table 30

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Kelson, La., Site

Factor	Line B 1950 Flood	Line B Project Flood	Design Values
River stage (crest)	48.7	53.7	53.7
Average el of ground or tailwater	32.0	32.0	32.0
Head on levee (H)	16.7	21.7	21.7
Piezometers used in analysis	A-1, -2, -3	----	----
Riverside borrow pit, width, ft	300	----	----
Top stratum*	15 ft clay 20 ft Sd Si	----	----
Average stratum**	15 ft clay	----	15 ft clay
Distance from riverside levee toe to river (L_1)†	1000	----	----
Base width of levee (L_2)	180	----	----
Landward extent of top stratum (L_3)††	400 1000‡	---- ----	---- ----
Distance to effective seepage source (s)	250	----	----
Effective length of riverside blanket (x_1)	70	----	----
Distance to effective seepage exit (x_3)	50	20	50
Effective thickness of sand substratum (d)††	6 90‡	---- ----	6 90
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	5	5	5
Laboratory permeability tests††	0.6	----	----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	----	----	----
Seepage and piezometric data	----	----	----
Field pumping tests	----	----	----
Well flow and piezometric data	----	----	----
Top stratum, type	Si Cl	----	----
Effective thickness for seepage analysis (z_{bL})‡‡	2	----	----
Critical thickness (z_t)	2	----	----
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)††	0.025	0.15	----
Piezometric data and blanket formulas	0.024	0.15	----
Piezometric data and seepage measurements	----	----	----
Permeability ratio (k_f/k_{bL})††	200	33	200
Blanket formula	208	33	----
Natural seepage measurements	----	----	----
Natural seepage beneath levee			
Q_s, gpm/100 ft of levee††	0.25	0.35	----
Q_s/H, gpm/ft of head/100 ft of levee	0.015	0.016	----

* Above sand foundation. Sandy silt stratum under levee exposed.
** In borrow pit.
† To sand bar.
†† For sandy silt stratum.
‡ For deep sand stratum.
‡‡ Above sandy silt top stratum.

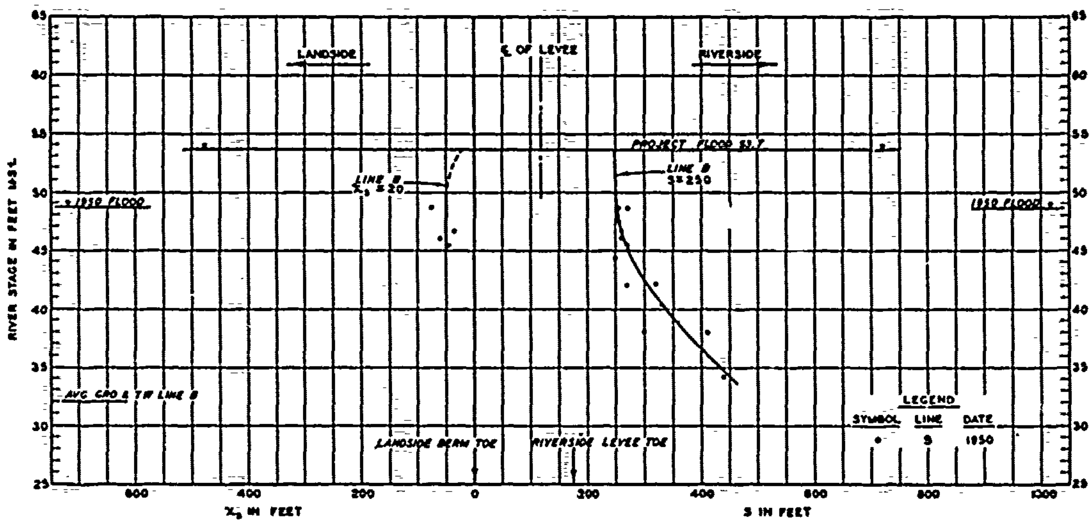

Fig. 42. Distances to effective seepage source and exit.
Kelson, line B

553. Thickness and permeability of silt stratum. The natural levee deposit of sandy silt underlying the levee at Kelson has numerous discontinuities and is quite variable in thickness; it has an average thickness of about 6 ft. The gradations of samples of this silt are plotted on plate 211. Laboratory tests on remolded samples of the sandy silt gave a permeability of 0.6×10^{-4} cm per sec; however, it is estimated that the horizontal permeability of this stratum in situ is about 5×10^{-4} cm per sec.

554. Thickness and permeability of top stratum. The top stratum of silty clay over the sandy silt at line B is only 1 to 2 ft thick. At line A the sandy silt stratum is only 2 to 3 ft thick and is not overlain by any impervious top stratum (see plates 204 and 205). The deeper and somewhat thicker sandy silt at line A does not extend landward of the levee. Although estimated values of the permeability of the top stratum at line B are shown in table 30 they are not considered to be reliable.

555. Permeability ratio. The ratio of permeability of the sandy silt stratum to that of the top stratum at piezometer line B at the crest of the 1950 high water was estimated to have been 200; however this value is not considered very reliable.

556. Seepage flow. Seepage passing through the sandy silt stratum

beneath the levee at line B was estimated for the crest of the 1950 high water, and for the project flood, using corresponding values of H, s, and x_3 for these floods (see table 30). The natural seepage passing beneath the levee at the 1950 crest was estimated to be about 0.25 gpm per 100 ft of levee; at project flood stage the seepage would probably increase to about 0.35 gpm. Q_s/H was estimated as being only 0.015 gpm per 100 ft of levee. Thus the amount of seepage passing through the natural levee deposits under the levee is extremely small. On the basis of the hydraulic gradient line shown on plate 209, practically all the seepage passing beneath the levee apparently emerges immediately landward of the toe of the road.

557. Landside substratum pressures. Hydrostatic pressures developing at and landward of the toe of the levee during the 1950 high water are shown for line B on plate 209. A plot of the estimated pressure in the sandy silt stratum at the landside toe of the road at Kelson is shown on plate 210; a similar plot of the pressure in the deep underlying sands as measured by piezometer A-5 vs river stages is also shown. The head on the levee, top stratum characteristics, substratum pressures, and the gradient through the top stratum at certain typical piezometers are given in table 31.

558. Uplift pressures that developed during the 1950 high water were not great enough in either the sandy silt or the underlying deeper

Table 31

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Kelson, La., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (43.7): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (53.7): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
B-D	A-3	34.2	----	3.8	0.0	3.8	3.8	3.2	14.5	0.0	0	----	----	----	----	19.5	----	--	----
B-D	A-4	34.2	----	0.0	10.0	10.0	10.0	8.5	14.5	0.0	0	----	----	----	----	19.5	----	--	----
B-D	A-3[e] and A-6[e]	32.0[e]	----	1.5[e]	0.0[e]	1.5[e]	1.5[e]	1.3	16.7	0.6	4	----	----	----	0.40	21.7	1.3[c]	6	17.3
B-D	A-6	32.8	----	1.8	2.0	3.8	3.8	3.2	15.9	0.0	0	----	----	----	----	20.9	----	--	----
B-D	A-7	32.8	----	8.0	1.5	1.5	8.0	6.8	15.9	0.0	0	----	----	----	----	20.9	----	--	----
B-D	A-5[f]	32.0[g]	----	12.0[g]	10.0[g,h]	22.0[g]	22.0[g]	18.7	16.7	4.6	28	----	----	----	0.21	21.7	8.9[a]	41	>21.7

a, c See paragraph 143.
e Average of A-3 and A-6.
f Piezometer in deep sand.
g Sixty feet landward of piezometer.
h Silt on top of clay.

sands to cause any sand boils landward of the levee. The thickness and permeability of the natural levee silts underlying the levee are not great enough to permit development of sufficient seepage flow and excess head to cause sand boils. Only light seepage and possible softening of the ground at the toe of the levee is expected to occur at the site. In view of the information derived from this investigation and the fact that no seepage or sand boils were observed either along the levee toe or in the fields at the crest of the 1950 high water, which was only 1 ft lower than the maximum stage experienced during the 1937 high water, it is difficult to understand how the heavy underseepage reported along this reach could have occurred.

559. Considerable excess head did develop beneath the thick clay stratum at the Kelson site as measured by piezometer A-5. A plot of readings of this piezometer vs river stage (plate 210) indicates that the maximum head that may be expected to develop beneath the clay stratum at project flood stage is about 9 ft. It is believed that the thick clay top stratum at line B can withstand an excess head of about 15 ft with safety.

Evaluation of seepage problem and recommendations for control measures

560. The criteria used for evaluating the safety of the levee against uplift are not considered applicable to the sandy silt stratum beneath the levee. For such a condition the safety of the levee against piping probably can best be evaluated from consideration of the creep ratio as described in paragraph 111. According to Bligh,[4] the creep ratio for a continuous stratum of very fine sand or silt should equal or exceed 18; Lane[22] recommends a weighted ratio of 8.5. At line B, C has a value of about 13 or 14. Considering the fact that along most of the Kelson site the sandy silt stratum beneath the levee is either quite thin or noncontinuous, it is doubtful that seepage through this stratum presents any significant danger to the levee.

561. On the basis of field observations and piezometer readings obtained at the crest of the 1950 high water, no additional seepage control measures are recommended for the Kelson site. Also no relief of

substratum pressures is needed in the sand foundation.

Baton Rouge, Louisiana

562. The site at Baton Rouge was selected for study because of the serious sand boils that occurred during the 1937 high water. Rather deep borrow pits have been excavated between the levee and river, but they do not penetrate the underlying pervious foundation because of the thickness of the top stratum.

Description of site

563. The site is located on the east bank of the Mississippi River a short distance south of Baton Rouge, La., and lies principally between the river and Louisiana State University. The levee at this site is located only about 600 ft from the river and has a net height of 23 ft. The site extends from about levee sta 55 to 110. Plans of the site, river, borrow pits, surface geology, and piezometers are shown on plate 212; plate 213 is an aerial mosaic of the site. No contour maps of the site were made because the terrain landward of the levee has practically no relief, varying only a foot or so in elevation over the entire area. River stages at the piezometer site can be estimated from the river gage at Baton Rouge and the graph on plate 211.

564. History of underseepage. During the 1937 high water (maximum H = 19 ft), 8 large sand boils occurred 200 to 4000 ft landward of the levee between sta 74 and 115. It was determined subsequently that these boils occurred at locations of seismic shot points made in connection with geophysical surveys. Locations of most of these sand boils are plotted on plate 212 and photographs of the boils are shown on plate 211.

565. During the 1945 high water (maximum H = 20 ft), underseepage occurred in numerous spots from the levee toe for a distance 5000 ft landward between sta 74 and 115. Four sand boils were observed.

566. During the 1950 high water (maximum H = 17.4 ft), some underseepage was observed emerging in drainage ditches in the fields and along the roads over an area extending 1/2 mile landward of the levee between sta 74 to 115. Four fairly large sand boils occurred at the crest of the

high water. However, according to available records, these boils were not at the same locations as the 1937 boils. Seepage measured between sta 74+25 and 98+00 on 15 March with an H of approximately 15 ft was 9 gpm per 100-ft station, or $Q_A/H = 0.6$ gpm. Excess hydrostatic pressures of 12.5 to 15 ft existed in the pervious substratum sands along the landside toe of the levee during the crest of the 1950 high water. These pressures represent heads of 75 to 90 per cent H . Excess heads of 10 to 12 ft were observed as far as 3/4 mile landward of the levee.

567. No underseepage control measures have been constructed at the Baton Rouge site.

568. Piezometer installation. The piezometer installation at Baton Rouge was largely made in 1942 and 1946, and consists essentially of four lines of piezometers perpendicular to the levee at the following stations:

Piezometer Line	Levee Station
A	73+40
B	87+87
C	97+42
E	107+66

Numerous other piezometers were installed along the landside toe of the levee (line G) from about sta 55 to 100; other piezometers were installed in the general area as shown on plate 212. Piezometer line C extends a distance of approximately 6000 ft landward of the levee. The tips of most of the piezometers were set near the top of the pervious substratum. However, as may be seen from the various soil profiles, a number of piezometer tips were set at various elevations in the top stratum to measure the distribution of head within the top stratum.

Geology of site and soil conditions

569. The general geology of the site is illustrated on plate 212. The principal area of investigation is located mainly on thick point bar deposits laid down during former river courses 7 through 9. As indicated on plates 212 and 214, a massive clay-and-silt-filled channel lies landward of the site between the boundary of course 7 and the terrace.

570. The fine-grained top stratum at the site is very heterogeneous in type of soil and thickness. Essentially it consists of an upper stratum of clay silts and silty clays approximately 10 to 15 ft thick underlain

by silts interspersed with numerous thinner strata of sands and clays, to a depth of 20 to 35 ft. These lower deposits of silts are frequently interspersed with irregular strata of fine sands and clays. The tops of some of the buried sandy silt and silty sand ridges are within 5 to 10 ft of the ground surface in some locations. In some areas within the site the total thickness of the top stratum is as much as 50 ft. Profiles of soil conditions are shown on plates 214-217.

571. It is interesting to note that the swale fillings generally do not extend below the base of the fine-grained top stratum, and for this reason several shallow swales visible on plates 212 and 213 have not been delineated on the soil profile sheets. The alignment of ridges and swales is at an angle of approximately 45° to the levee. The thick clays encountered beyond the northeast margin of course 7 represent back-swamp deposits laid down in deep depressions bordering the Prairie terrace (represented by the white areas on the right-hand side of plate 212). Up to 10 ft of predominantly clayey natural levee deposits, which cannot be distinguished from the underlying top stratum materials with respect to grain size, cover much of the site. The height of the natural levee along the present course of the Mississippi River bank is clearly illustrated on plates 214 and 215.

572. Relation of underseepage to geology. Underseepage at the site appears in general to be related to irregularities in the thickness of the top stratum. According to Fisk,[11] sheet seepage is confined to subsurface sandy ridges between swales; seepage through swales is practically unknown. In some reported instances during previous high waters at Baton Rouge areas between swales became sufficiently soft to bog down farm animals. Such conditions probably were a result of excessive substratum pressures and concentration of seepage rising through the ground to the surface in areas where sandy ridges extend almost to the surface.

573. The sand boils occurring in this area during the 1937 flood ranged in size from minute "crayfish boils" associated with areas of general seepage to several large boils which required sacking. The larger sand boils were studied by Fisk, and he found that, without exception, all occurred near ridges in the top stratum. Fisk believes that

development of most of the sand boils was aided, if not initiated, by poorly backfilled seismic shot holes. The largest boil occurred at a distance of about 600 ft from the levee toe; however, boils actually developed at locations as far as a mile or two from the levee.

574. The top stratum at the site is sufficiently thick and impervious to permit the development of high substratum pressures throughout the entire area. With such a high substratum pressure prevalent over the entire site, the location of underseepage and sand boils is particularly affected by irregularities and weak spots in the top stratum resulting from either artificial or natural occurrences. In this connection it is pointed out that no sand boils occurred at the site prior to the 1937 flood (H = approximately 20 ft) or prior to the period (1935-1937) when geophysical parties were active in tracing the limits of the LSU oil field. Considerable general seepage but no large sand boils occurred during the 1927 flood (maximum H = 22.8 ft). Only relatively minor sand boils occurred during either the 1945 or 1950 high waters.

575. Soil profiles and piezometer lines. Soil profiles and piezometer lines, both perpendicular and parallel to the landside toe of the levee, are shown on plates 214 to 217. In the initial layout of the installation most of the piezometers were located along the landside toe of the levee and along two existing roads (lines A and B) which are more or less perpendicular to the levee. A number of individual piezometers were installed at locations where large sand boils had occurred during the 1937 high water. Subsequent to the initial installation a number of additional piezometers were installed at the site to permit better delineation of subsurface pressure conditions beneath and within the top stratum.

576. The top stratum has been described under "Geology of site and soil conditions."

577. The pervious substratum in the Baton Rouge area was not explored below a depth of about 65 ft. From other boring data available it appears that the sand stratum extends to a depth of about 210 ft, which would make the thickness of the sand aquifer approximately 175 ft. On the basis of information available the upper 30 ft of the sand varies

from very fine to coarse. Little is known about the deeper portion except that it extends out to the channel of the Mississippi River. There is no information regarding the permeability of the pervious aquifer at the site.

Analysis of piezometric and seepage data

578. River stages, piezometer readings at lines A and C, and rainfall data observed at the Baton Rouge site during the entire year of 1945 are plotted on plates 218 and 221; other piezometer data for the 1945 high-water period are shown on plates 219, 220, and 222. River stage and piezometer readings during the 1950 high water are plotted on plates 223 through 226. At the crest of the 1945 and 1950 high waters, H was about 20 and 17 ft, respectively. The following analysis of data primarily pertains to conditions observed in 1950.

579. Piezometric gradients in the pervious substratum beneath and landward of the levee at piezometer lines A, C, and E are shown on plates 227-229 for selected river stages during the 1945 and 1950 high waters. The hydrostatic head along the toe of the levee as measured by piezometers along line G is shown on plate 230. These plates show that excess heads of about 15 to 16 ft developed at the levee toe throughout the entire reach at the crest of the 1950 high water. Hydrostatic heads above the ground surface of about 10 to 12 ft were observed as far as 3000 ft landward from the levee (see plates 227-229).

580. A summary of information pertaining to the site and results of analyses of piezometric and seepage data are given in table 32.

581. Source of seepage. Values of s determined at piezometer lines A and C during the 1950 high-water period are plotted in fig. 43; because of insufficient data for piezometer line E, s was not determined at this line. The values of s shown in fig. 43 and plates 227 and 228 indicate that seepage enters the sand substratum through the bank and bed of the Mississippi River. A certain scattering in the piezometric data was apparent, which affects the determined values of s as shown in fig. 43. A slight decrease in s occurred during initial rising stages; at the crest of the 1950 flood s was about 800 ft at lines A

Table 32

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Baton Rouge, La., Site

Factor	Line A: 1950 Flood	Line A: Project Flood	Line C: 1950 Flood	Line C: Project Flood	Design Values: Sta 54-65	Design Values: Sta 65-70	Design Values: Sta 70-106
River stage (crest)	42.4	48.0	42.4	48.0	48.0	48.0	48.0
Average el of ground or tailwater	25.0	25.0	25.0	25.0	25.0	25.0	25.0
Head on levee (H)	17.4	23.0	17.4	23.0	23.0	23.0	23.0
Piezometers used in analysis	7, 9, & 11	----	21C, 22A & 23	----	----	----	----
Riverside borrow pit, width, ft	300	----	200	----	250	300	200
Top stratum	33 ft Si & Cl	----	15 ft Cl & 7 ft silt	----	30 ft Cl & Si	30 ft Si & Cl	25 ft Cl & Si
Average stratum	25 ft Cl	----	20 ft Cl	----	25 ft Cl	25 ft Cl	20 ft Cl
Distance from riverside levee toe to river (L_1)	500	----	500	----	500	500	500
Base width of levee (L_2)	210	----	210	----	210	210	210
Landward extent of top stratum (L_3)	3200	----	5000	----	1500	3200	5000
Distance to effective seepage source (s)	850	850	800	800	850	850	800
Effective length of riverside blanket (x_1)	640	640	590	590	640	640	590
Distance to effective seepage exit (x_3)	10,000	10,000	11,200	11,200	14,000	12,000	11,000
Effective thickness of sand substratum (d)	175	----	175	----	175	175	175
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	500	----	500	----	500	500	500
Laboratory permeability tests	----	----	----	----	----	----	----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	----	----	----	----	----	----	----
Seepage and piezometric data	600	----	325	----	----	----	----
Field pumping tests	----	----	----	----	----	----	----
Well flow and piezometric data	----	----	----	----	----	----	----
Top stratum, type	Si & Cl	----	Si & Cl	----	Cl & Si	Si & Cl	Si & Cl
Effective thickness for seepage analysis (z_{bL})	30	----	30	----	25	30	30
Critical thickness (z_t)	30	----	30	----	25	30	30
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	0.06	0.07	0.06	0.06	----	----	----
Piezometric data and blanket formulas	0.09	0.09	0.06	0.06	----	----	----
Piezometric data and seepage measurements	0.05*	----	0.05*	----	----	----	----
Permeability ratio (k_f/k_{bL})	8350	7000	9000	9000	5000	7000	9000
Blanket formula	5450	5450	9000	9000	----	----	----
Natural seepage measurements	10,000	----	10,000	----	----	----	----
Natural seepage beneath levee							
Q_s, gpm/100 ft of levee	20.6	27.3	18.7	24.7	----	----	----
Q_s/H, gpm/ft of head/100 ft of levee	12	1.2	1.1	1.1	----	----	----
Q_s/H, gpm/100 ft levee between sta 74+25 and 98+00 and between levee and 1/2 mile landward (measured)	----	----	0.52	----	----	----	----

* Based on total measured seepage, average thickness of top stratum, and average excess head beneath top stratum for area landward of levee where observed seepage was emerging.

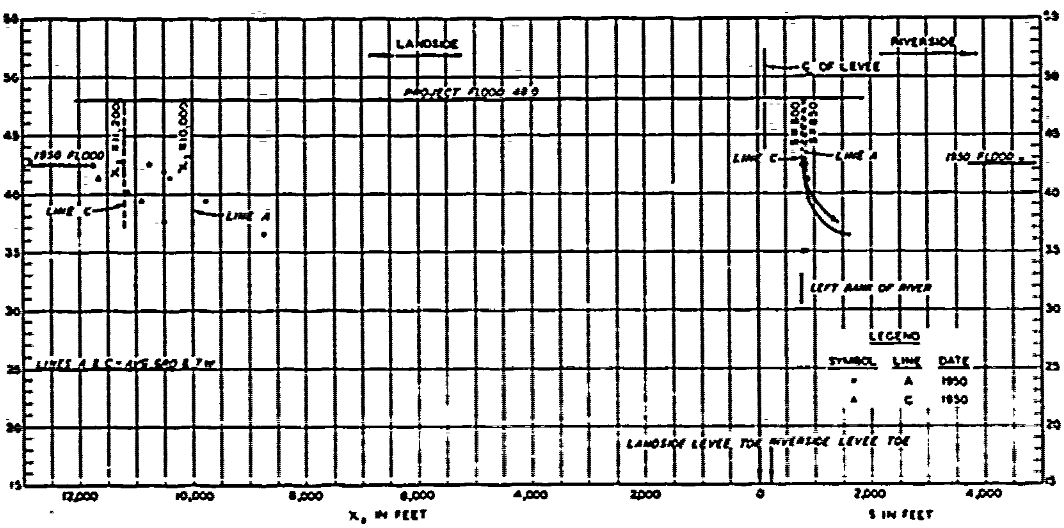

Fig. 43. Distances to effective seepage source and exit. Baton Rouge, lines A and C

and C. It is estimated that s will also be about 800 ft at these two piezometer lines at project flood stage (see fig. 43 and table 32).

582. Seepage exit. The values of x_3 for the 1950 high water are plotted vs corresponding river stages in fig. 43. At piezometer line A, where the top stratum consists of about 30 ft of silt and clay, x_3 was 10,000 ft, and at line C, where the top stratum has about the same thickness as at line A, x_3 was 11,200 ft at the 1950 crest. These values of x_3 are considerably greater than those obtained at the other sites studied, and may be attributed to the thick top stratum at the site and the fact that the weak spots caused by the seismic shots have apparently healed, at least to the extent that they can withstand river stages equal to those occurring during 1945 and 1950.

583. Thickness and permeability of substratum sands. Little is known regarding the exact depth and characteristics of the substratum sands. On the basis of nearby borings and geological information, it is estimated that the pervious foundation has an effective thickness of 175 ft. The permeability of the substratum was estimated from natural seepage measurements and piezometric data to be between 300 and 600 x 10^{-4} cm per sec. Results of these determinations are summarized in table 32. For analytical purposes it was assumed that $k_f = 500 \times 10^{-4}$ cm per sec at the site.

584. Thickness and permeability of top stratum. The average top stratum landward of the levee is considered to have an effective thickness of about 30 ft; see plates 214-217. However, it should be noted that there is considerable variation in the character and thickness of the top stratum landward of the levee. On the basis of the top stratum thickness listed in table 32, k_{bL} computed from piezometric data was found to be about 0.06 to 0.09 x 10^{-4} cm per sec. The permeability of the top stratum from measurements of natural seepage was estimated to be 0.05 x 10^{-4} cm per sec. The permeability of the top stratum at the Baton Rouge site is believed equal to about 0.06 x 10^{-4} cm per sec. This comparatively low permeability probably can be attributed to the greater extent and thickness of clays and silts at this site as compared to those at other piezometer locations.

585. Permeability ratio. The ratio of the permeability of the foundation to that of the top stratum was estimated at piezometer lines A and C to have been about 8400 and 9000, respectively, at the 1950 flood crest. Similar estimates of k_f/k_{bL} for the project flood are given in table 32.

586. Seepage flow. Seepage passing beneath the levee at lines A and C at the crest of the 1950 high water, and for the project flood, was estimated using corresponding values of H, s, and x_3 for these floods (table 32). The estimated seepage at the 1950 crest was about 20 gpm per 100 ft of levee, or $Q_s/H = 1.2$ gpm. Q_A/H as determined from seepage measurements during this high water was about 0.5 gpm. In comparing the above values of Q_s/H and Q_A/H it is pointed out that Q_s/H, as determined from piezometric data and foundation permeability characteristics, represents the total seepage flow passing beneath the levee, whereas the value for Q_A/H includes only the seepage emerging between the levee and the landward edge of the seepage-measuring point landward of the levee. Based on the piezometric and seepage data obtained during the 1950 high water, about 50 per cent of the above-computed Q_s was emerging landward of the levee in the seepage-measuring area. From these data it may be concluded that the Baton Rouge site is not subject to a high rate of natural seepage and even during a project flood stage the total quantity of seepage passing beneath the levee will probably be quite small.

587. Landside substratum pressures. The hydrostatic pressures which developed along the toe of the levee at or near the crest of the 1950 flood are shown on plate 230 (line G). Readings of selected piezometers at or near the landside toe of the levee vs river stages are plotted on plate 231. Also shown are estimated substratum pressures for river stages up to project flood stage. The head on the levee, top stratum characteristics, and observed and computed maximum substratum pressures at certain piezometers along the landside toe of the levee or landward of the levee are given in table 33.

588. Plates 218-226 show that changes in river stage were quickly reflected in the piezometers installed in the foundation sand. However, minor irregularities in the readings of the deep piezometers occasionally

Table 33

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Baton Rouge, La., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, mGl	Est Tailwater el, ft mGl	Thickness of Top Stratum, ft: Clay	Silt	Total	z_t	h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (42.4): H ft	h_o ft	$\frac{h_o}{H}$ %	Est Gradient through Top Stratum (1950 Flood): Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	Project Flood (48.0): H ft	h_o ft	$\frac{h_o}{H}$ %	Est H at i_c ft
G	P-37	26.0	----	----	----	24.2	24.2	20.6	16.4	---	--	----	----	----	----	22.0	----	--	----
G	P-1	25.4	----	15.5	9.5	25.0	25.0	21.2	17.0	6.5	38	----	----	0.26	----	22.6	----	--	----
G	P-1-A	25.4	----	15.5	20.5	36.0	36.0	30.6	17.0	14.5	85	----	----	0.40	----	22.6	----	--	----
G	P-2	26.4	----	21.0	17.5	38.5	38.5	32.6	16.0	3.0	19	----	----	----	0.08	21.6	4.8[a]	22	>21.6
G	P-3	26.4	----	17.0	4.0	21.0	17.0	14.5	16.0	3.3	21	----	----	----	0.19	21.6	5.8[a]	27	>21.6
I	P-4	24.1	----	----	----	20.0	20.0	17.0	18.3	----	--	----	----	----	----	23.9	----	--	----
A-G	P-5	25.5	----	----	----	51.0	51.0	43.4	16.9	12.1	72	----	----	0.24	----	22.5	14.5[a]	64	>22.5
A-G	P-6-A	25.0	----	10.0	3.0	13.0	13.0	11.0	17.4	----[e]	--	----	----	----	----	23.0	----	--	----
A-G	P-10	23.8	----	6.0	4.0	10.0	10.0	8.5	18.6	----	--	----	----	----	----	24.2	----	--	----
A-G	P-11-B	23.0	----	7.0	0.0	7.0	7.0	6.0	19.4	1.9	10	----	----	0.27	----	25.0	----	--	----
A-G	P-11	23.0	----	[illegible]	18.0	25.0	25.0	21.2	19.4	15.0	77	----	----	0.60	----	25.0	19.8[a]	79	>25.0
G	P-32-A	25.5	----	20.0	3.0	23.0	20.0	17.0	16.9	0.8	5	----	----	----	0.04	22.5	2.0[a]	9	>22.5
G	P-33	25.5	----	18.0	22.5	40.5	40.5	34.4	16.9	----	--	----	----	----	----	22.5	----	--	----
G	P-35	26.0	----	----	----	32.5	32.5	27.6	16.4	----	--	----	----	----	----	22.0	----	--	----
G	P-13	26.0	----	----	----	27.0	27.0	23.0	16.4	15.2	93	----	----	0.56	----	[illegible]	[illegible]	92	>22.0
G	P-31-A	25.5	----	18.0	17.0	35.0	35.0	29.8	16.9	10.9	64	----	----	0.31	----			--	----
G	P-18	26.2	----	----	----	31.0	31.0	26.4	16.2	15.2	94	----	----	0.49	---	[illegible]	[illegible]	95	>21.8
C-G	P-19	25.5	----	31.5	2.0	33.5	31.5	26.8	16.9	12.4	73	----	----	0.39	----	22.5	[illegible]	68	>22.5
C-G	P-21-A	24.8	----	10.0	0.0	10.0	10.0	8.5	17.6	0.3	2	----	----	----	0.03	23.2	----	--	----
C-G	P-21	24.8	----	----	----	22.5	22.5	19.1	17.6	1.2	7	----	----	----	0.05	23.2	----	--	----
[illegible]	P-45 and P-23	23.5	----	----	----	31.0	31.0	26.4	18.9	16.9	89	----	----	0.55	----	24.5	21.6	88	>24.5

[a] See paragraph 143.

[e] [illegible] piezometer readings did exceed ground surface elevation.

were noted. The deeper piezometers installed in the top stratum also quickly reflected changes in river stage; the shallow piezometers, however, appear to be affected more by rainfall than by the river stage (see piezometers 6-A, 11-B, and 21-A; plates 218 and 221).

589. From the data shown on plate 231 and in table 33 it appears that uplift pressures sufficient to cause sand boils will not develop along the levee toe at the Baton Rouge site during the project flood unless the old shot holes become active. Although minor boils were observed during the 1945 and 1950 high-water periods under heads of about 17 to 20 ft on the levee these boils were comparatively small. The observed uplift pressures in 1950 correspond to about 75 to 95% H. The maximum gradient through the total top stratum observed during the 1950 high water was about 0.6. In general, the gradients through the top stratum ranged from about 0.2 to 0.5. In summary, although excess heads as high as 20 ft may develop at the crest of a project flood stage, the top stratum appears to have sufficient thickness to withstand a head of this amount. A few sand boils have occurred at the site with river

stages producing an H of about 20 ft, but as H for the project flood will be only about 23 ft the site is not considered critical with respect to underseepage. It is pointed out, however, that in view of the unusually high excess hydrostatic heads that develop landward of the levee some sand boils may be expected to recur at thin or weak spots during high river stages.

590. Hydrostatic head vs depth. Several piezometers were installed at various elevations within the top stratum to determine the distribution of head with depth as the subsurface seepage rose to the surface. The piezometric head above ground surface as measured by piezometers at various depths in lines A and C at about the crest of the 1950 high water (H = about 17 ft) is plotted in fig. 44. The data in this figure indicate that the hydrostatic head beneath the top stratum was about 12 to 15 ft and that this head decreased rapidly as a result of seepage flowing upward through the lower 10 to 20 ft of the top stratum. It is interesting to note that the rate of head loss through the top stratum at the points of measurement was about as great through the silty sand and sandy silt strata as through the clay strata. This is difficult to explain, as

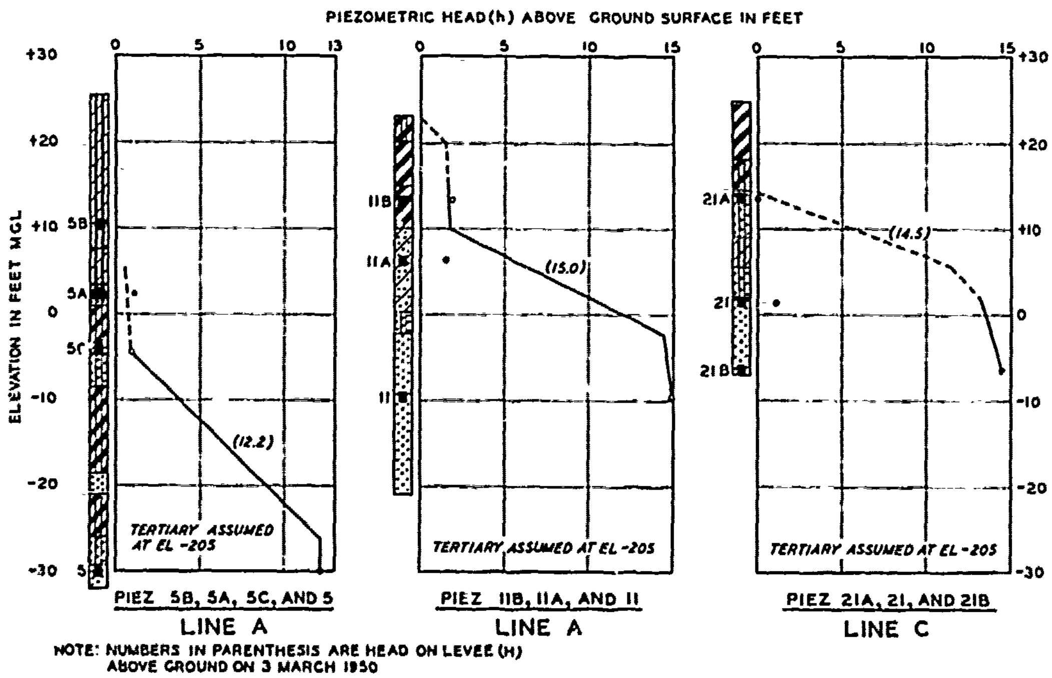

Fig. 44. Piezometric head landward of levee vs depth. [illegible] ..on Rouge, La.

the deep clay strata are rather thick and probably are not perforated by shrinkage cracks, root holes, or crayfish holes. However, on the basis of the data shown in fig. 44 it appears that the deep silty soils are no more permeable than the clayey soils at this site.

Evaluation of seepage problem and recommendations for control measures

591. An H of about 17 ft during the 1950 high water created four sand boils at the site. Several sand boils also developed during the 1937 and 1945 high waters. The sand boils that developed in 1937 are attributed to weak spots in the top stratum resulting from seismic shots made in connection with geophysical surveys. The 1945 sand boils were less severe and apparently developed at thin spots in the top stratum. However, since then the weak or thin spots seemed to have largely healed as the boils that occurred in 1950 were at different locations. As the levee at Baton Rouge has withstood heads within about 3 ft of those that will exist at project flood stage with only a few sand boils developing, the levee is considered safe as regards underseepage. The hydrostatic heads landward of the levee predicted for the project flood will not be sufficient to create sand boils except in areas where discontinuities in the top stratum may exist and possibly where boils previously have developed. The top stratum at the site is comparatively thick, although somewhat irregular in character. Although some minor boils may be expected at project flood stage, no seepage control measures are considered necessary.

Cotton Bayou, Louisiana

592. The Cotton Bayou site was selected for investigation because it is along a reach of levee on the Red River where, prior to construction in 1947 of the present setback levee, the former levee had been subjected to considerable underseepage during high-water periods.

Description of site

593. The site is located between Alexandria Municipal Airport and the Red River approximately 8 miles upstream from Alexandria, La. The

site as discussed in this report extends from about sta 815 to 865. Plans of the site, river, borrow pits, surface geology, topography, and piezometers are shown on plates 232-234; plate 235 is an aerial mosaic of the site. The levee at Cotton Bayou has a net height of approximately 17 ft for the interim flood; the net height for the project flood is about 13.5 ft.* Rather extensive riverside borrow pits approximately 300 ft wide and 15 ft deep have been excavated along the portion of the levee studied; in some of these pits the natural top blanket of clay has been removed and the underlying silts have been almost penetrated. Although a few feet of head was created on the levee during 1950, it has not yet been subjected to a significant high water. River stages at the Cotton Bayou site can be estimated from the Alexandria, La., gage and the graph on plate 243.

594. History of underseepage. There is no history of underseepage to date for the present levee as there has been no high water of any significance since construction of the setback levee.

595. Piezometer installation. In 1948 two lines of piezometers (B and D) were installed perpendicular to the levee at sta 826+49 and 842+90, with some (line A) placed along the berm between sta 820 and 850 (plates 233-235). Piezometer readings were obtained during the 1950 high water.

Geology of site and soil conditions

596. The general geology of the site is illustrated on plate 232; the type and thickness of the top stratum soils are illustrated in more detail on plates 233 and 234. The site is located mainly on natural levee, backswamp, and channel deposits laid down as the Red River shifted its course from the position marked by Bayou Rapides to essentially its present position. Cotton Bayou follows a former channel of Red River filled by thick clay and silt deposits (plates 233, 237, 238). Big Bayou may also follow a former course of the Red River, but the data available are not sufficient to establish this possibility. In the area between Cotton Bayou and the present course of Red River, the top stratum

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* Project flood height after completion of authorized reservoirs.

consists of 5 to 10 ft of clay silts underlain by 5 to 10 ft of silty sands and sandy silts (see plates 236 and 237). A deep clay-and-silt-filled swale extends roughly parallel to the river on the riverside of the levee (plates 233 and 236). Silts and sands are exposed in the bottom of the borrow pits. Thin, predominantly clayey natural levee deposits may cover much of the site, but if so they are insignificant in comparison with similar materials bordering the old Bayou Rapides course of the Red River and for this reason they have not been delineated on plate 232. Backswamp clays covering what are probably point bar deposits occupy an extensive area south and west of the site.

597. Soil profiles and piezometer lines. Soil profiles and piezometer lines both perpendicular and parallel to the levee are shown on plates 236-238. Piezometer line B was located perpendicular to the levee at a point considered representative of the general area bounded by the levee on the north and Cotton Bayou. Line D was located perpendicular to the levee at a point considered potentially critical as regards underseepage because of the nearness of a thick clay deposit immediately landward of the levee and of riverside borrow pits penetrating to the top of the underlying pervious sands. As may be seen from the soil profiles on plates 236-237, the tips of some of the piezometers were installed both in the top of the substratum sands and in the silty sands immediately beneath the upper top stratum of clay silt. Piezometer line D extends from the abandoned levee near the present position of the Red River to a distance of approximately 1500 ft landward of the levee.

598. The sediments making up the top stratum in the area between Cotton Bayou and the levee are very uniform as regards thickness and type. As previously stated, the top stratum consists of 5 to 10 ft of clayey silts underlain by about 5 to 10 ft of silty sands and sandy silts. The filling in the former course of Red River presently marked by Cotton Bayou consists of about 20 to 30 ft of clay. The clay-filled swale between the levee and Red River is filled with alternating strata of clays and silts to a depth of approximately 35 ft. Although the area landward of the levee is relatively uniform in elevation there is a perceptible slope along the toe of the levee down to the former course of the Red

River which is now filled with sediments and occupied by Cotton Bayou. The pervious substratum at the site consists of about 30 to 40 ft of fine sands (see plate 236). It appears on the basis of boring 1-X that the top of Tertiary may be as high as el 40 beneath the abandoned levee. This in effect reduces the thickness of the pervious aquifer between the present levee and Red River to about 10 to 25 ft.

Analysis of piezometric and seepage data

599. River stages and piezometer readings observed at Cotton Bayou during the 1950 high water are plotted on plate 239. This high water created a maximum H of about 5 ft, or 12 ft below the interim flood stage. Piezometric gradients beneath the levee along piezometer lines B and D perpendicular to the levee are shown on plates 240 and 241, respectively. The hydrostatic head along the toe of the levee (line A) is shown on plate 241. From plates 240 and 241 it is apparent that no hydrostatic head developed above the ground surface during the 1950 high water. A summary of information pertaining to the site is given in table 34.

600. Source of seepage. Because of the configuration of the piezometric grade line beneath the levee (plates 240 and 241), s could not be determined as was done for the other sites. The data obtained at lines B and D show that ground-water storage still was being filled at the crest of the 1950 flood, and a portion of the semipervious strata under the levee had not yet become saturated. Although it appears from plate 240 that the point of seepage entry may have been about 900 ft riverward of the levee on 24 February 1950, this point may not be indicative of the true value of s, which can be obtained from piezometer data only when artesian flow conditions exist beneath the levee. Seepage may enter the sand substratum at the site through the bank of Red River and, to an unknown extent, through the bottom of riverside borrow pits (see plates 236 and 237).

601. Seepage exit. Values of x_3 could not be determined, as no head developed above the ground surface landward of the levee during the 1950 high water.

602. Thickness and permeability of substratum sands. The sand

Table 34

Summary of Analysis of Piezometric and Seepage Data, and Average Design Values

Cotton Bayou, La., Site

Factor	Line B 1950 Flood	Line B Interim Flood	Line D 1950 Flood	Line D Interim Flood	Design Values Sta 817-830	Design Values Sta 830-836	Design Values Sta 836-848	Design Values Sta 848-850
River stage (crest)	86.5	99.0	86.5	99.0	99.0	99.0	99.0	99.0
Average el of ground or tailwater	82.0	82.0	81.5	81.5	85.0	82.0	81.0	80.0
Head on levee (H)	4.5	17.0	5.0	17.5	14.0	17.0	18.0	19.0
Piezometers used in analysis	----	----	----	----	----	----	----	----
Riverside borrow pit, width, ft	225	----	300	----	250	250	250	250
Top stratum	0-5 ft Cl Si 5-15 ft Sd Si	----	1-3 ft Cl	----	Si Sd	Si Sd	Si Sd	Si Sd
Average stratum	10 ft Sd Si	----	2 ft Cl	----	6	8	5	3
Distance from riverside levee toe to river (L_1)	1050	----	1600	----	1000	1300	1600	1600
Base width of levee (L_2)	200	----	225	----	200	225	225	250
Landward extent of top stratum (L_3)	2300	----	350	----	1600	2200	350	100
Distance to effective seepage source (s)	----	----	----	----	1000	1000	1000	1000
Effective length of riverside blanket (x_1)	----	----	----	----	800	775	775	750
Distance to effective seepage exit (x_3)	----	----	----	----	160	460	800	2100
Effective thickness of sand substratum (d)	30	----	30	----	30	30	30	30
Permeability of substratum ($k_f \times 10^{-4}$ cm/sec)	200	----	200	----	200	200	200	200
Laboratory permeability tests	30	----	30	----	----	----	----	----
Grain size ($k_{f(field)}$ vs D_{10}, fig. 17)	200	----	200	----	----	----	----	----
Seepage and piezometric data	----	----	----	----	----	----	----	----
Field pumping tests	----	----	----	----	----	----	----	----
Well flow and piezometric data	----	----	----	----	----	----	----	----
Top stratum, type	Clay	----	Clay	----	Clay	Clay	Clay	Clay
Effective thickness for seepage analysis (z_{bL})	7.5	----	8.2	----	7.0	7.0	8.0	7.0
Critical thickness (z_t)	7.5	----	8.2	----	7.0	7.0	8.0	7.0
Permeability ($k_{bL} \times 10^{-4}$ cm/sec)	----	----	----	----	0.2	0.2	0.2	0.2
Piezometric data and blanket formulas	----	----	----	----	----	----	----	----
Piezometric data and seepage measurements	----	----	----	----	----	----	----	----
Permeability ratio (k_f/k_{bL})	----	----	----	----	1000	1000	1000	1000
Blanket formula	----	----	----	----	----	----	----	----
Natural seepage measurements	----	----	----	----	----	----	----	----
Natural seepage beneath levee								
Q_s, gpm/100 ft of levee	----	----	----	----	----	----	----	----
Q_s/H, gpm/ft of head/100 ft of levee	----	----	----	----	----	----	----	----

substratum at Cotton Bayou consists of about 30 ft of sand varying from very fine to medium fine. On the basis of grain size data, plate 243 and fig. 17, k_f would be about 200 x 10^{-4} cm per sec. Laboratory permeability tests indicated that the permeability of the sand stratum would be only 30 x 10^{-4} cm per sec. It is believed that k_f is probably about 200 x 10^{-4} cm per sec at this site. The low permeability may account for the comparatively long time required to fill ground-water storage landward of the levee during initial rising river stages.

603. Thickness and permeability of top stratum. The top stratum at Cotton Bayou consists of about 7 to 8 ft of clay underlain in turn by silty sands and sandy silts with a thickness of about 2 to 10 ft. It was not possible to determine the value of k_{bL} from piezometric data. Since the present levee has not been subjected to a high water and previous high waters against the abandoned riverfront levee probably did not cause significant, if any, seepage landward of the present setback

levee, the top stratum landward of the present levee probably has not yet been subjected to upward seepage. In view of this, k_{bL} at this site is believed to be somewhat lower than that at many of the other sites studied where boils and piping have developed through top strata of similar characteristics and thicknesses. Therefore it has been assumed that $k_{bL} = 0.2 \times 10^{-4}$ cm per sec.

604. Permeability ratio. On the basis of values given above, the ratio of permeability of the top stratum to that of the pervious substratum is estimated to be about 1000.

605. Seepage flow. Seepage passing beneath the levee could not be estimated from the piezometric data. Because of the comparatively low permeability of the substratum at this site, seepage beneath the levee will probably be small even during high floods.

606. Landside substratum pressures. Hydrostatic pressures along the toe of the berm during the 1950 high water are shown on plate 241 (line A). The tips of most of the piezometers along this line were set in the silty sands beneath the upper clay portion of the top stratum. As evidenced from these piezometers no excess head developed above ground surface during the 1950 high water. Selected piezometer readings vs river stages are plotted on plate 242. Also shown on this plate are estimated piezometer readings for river stages up to the interim flood. The head on the levee, top stratum characteristics, and estimated critical heads at certain typical piezometers are given in table 35.

607. Based on the data on plate 242 and in table 35, the Cotton Bayou site might possibly be critical as regards underseepage, as river stages of about 11 to 13 ft may produce excessive substratum pressures. The interim flood stage will result in an H of about 14 to 19 ft, with the possible development of critical uplift pressures. The most critical reach is probably at the crossing of the levee and Cotton Bayou in the vicinity of sta 850, where a discontinuity exists in the top stratum. However, in view of the limited data obtained to date, it is not possible to predict the severity of the seepage problem with any degree of certainty.

Table 35

Head on Levee, Top Strata, Substratum Pressures, and Gradients through Top Strata along Toe of Levee

Cotton Bayou, La., Site

Piez Line	Piez Number	Avg Gradient at Piez, el ft, msl	Est Tailwater el, ft msl	Thickness of Top Stratum, ft				h_c $(0.85\ z_t)$ ft	Crest of 1950 Flood (86.5)			Est Gradient through Top Stratum (1950 Flood)				Project Flood (99.0)[f]			Est H at i_c ft
				Clay	Silt	Total	z_t		H ft	h_o[e] ft	$\frac{h_o}{H}$ %	Sand Boils	Heavy Seepage	Med Seepage	Light or No Seepage	H ft	h_o ft	$\frac{h_o}{H}$ %	
A	A-1	85.0	----	6.5	---	6.5	6.5	5.5	1.5	---	--	----	----	----	----	14.0	----	--	----
B-A	B-5	82.7	----	7.5	---	7.5	7.5	6.4	3.8	---	--	----	----	----	----	16.3	----	--	----
B	B-6	81.0	----	7.5	2.0	9.5	7.5	6.4	5.5	---	--	----	----	----	----	18.0	----	--	----
A	C-1	81.8	----	7.5	3.0	10.5	7.5	6.4	4.7	---	--	----	----	----	----	17.2	----	--	----
D-A	D-3	81.0	----	8.2	4.5	12.7	8.2	7.0	5.5	---	--	----	----	----	----	18.0	7.0[g]	39	12.8[d]
D	D-5	80.0	----	4.0	0.0	4.0	4.0	3.4	6.5	---	--	----	----	----	----	19.0	----	--	----
D	D-6	80.0	----	4.0	6.7	10.7	10.7	9.1	6.5	---	--	----	----	----	----	19.0	----	--	----
A	E-1	80.0	----	7.0	---	7.0	7.0	6.0	6.5	---	--	----	----	----	----	19.0	6.0[g]	32	10.8[d]

[d] See paragraph 143.

[e] Head on levee not sufficient to create any excess head above ground surface landward of levee.

[f] Interim flood.

[g] Values obtained by extrapolating data on plate 242. As no excess head developed landward of the levee in 1950, the extrapolation of h_o to the interim flood crest is of questionable accuracy.

Evaluation of seepage problem and recommendations for control measures

608. Sustained river stages greater than 11 ft on the levee (about 6 to 7 ft below interim flood stage) may cause critical substratum pressures and the formation of sand boils at about sta 850 and in the reach upstream from sta 850. Downstream from sta 850 the top stratum appears sufficiently thick to withstand the substratum pressures expected to develop during the interim flood. On the basis of the 1950 piezometric data it is not possible to evaluate the adequacy of the existing landside berm or to determine definitely whether seepage control measures are required at Cotton Bayou site. Irregularities in the piezometric data make prediction of subsurface pressures at high river stages and the design of control measures somewhat difficult. However, it is suggested that abatis dikes be constructed across the existing riverside borrow pits to promote silting, thereby increasing the distance to the effective source of seepage; this measure alone may be adequate. The need for additional seepage control measures should be based on piezometric data obtained during future high-water periods.

PART V: EVALUATION OF DATA FROM PIEZOMETER SITES

609. This portion of the report consists of a general summary and evaluation of data obtained from the piezometer sites described in Part IV. Included in the evaluation are discussions of (a) the influence of geologic and man-made features on underseepage and sand boils; (b) permeability of riverside top strata; (c) source of seepage; (d) permeability of landside top strata; (e) seepage exit; (f) permeability of pervious substrata; (g) ratio of permeability of pervious substrata to landside top strata; (h) the relationship between upward gradients through top strata and severity of seepage; (i) the effect of natural partial cutoffs and massive clay deposits landward of levees on seepage; and (j) an evaluation of seepage berms for controlling underseepage.

610. A summary of soil conditions, analyses of piezometric and seepage data, and seepage conditions for a principal piezometer line at each site are given in table 36. Only the soil conditions and maximum head on the levee in 1950 are listed for the Cotton Bayou site, because the high water at this site during 1950 was not of sufficient duration to create truly artesian flow conditions, and as a result the piezometer data could not be analyzed.

Effect of Geologic and Man-made Features on Underseepage

Geology and seepage

611. Geologic and natural topographic features affect both the distribution and concentration of seepage landward of levees and, to some extent, the magnitude of substratum pressures. These effects are described in detail in Part II of this report; however, pertinent illustrations in terms of the 16 sites studied are given below.

612. Point bar deposits. The effect of point bar deposits on the location of seepage and sand boils is illustrated by the location of such at the Commerce, Trotters 51, Stovall, and Farrell sites. It can be seen from the plans of topography and surface geology at these sites that most of the sand boils occurred in ridges adjacent to swales. Examples of

Table 36

Summary of 1950 High-water Data at Piezometer Sites

Site	Station	Line	H 1950	H Project Flood	L_1 ft	L_2 ft	L_3 ft	Foundation d ft	Foundation $k_f \times 10^{-4}$ cm/sec	Top Stratum Soil Type*	Top Stratum z_{bL} ft*	Top Stratum $k_{bL} \times 10^{-4}$ cm/sec	Borrow Pits Blanket Soil Type*	Borrow Pits Thickness ft*	k_f/k_{bL}	Obs x_1 ft	s ft	x_3 ft	x_3, ft at River Stage at Which i = 0.85	h_o, ft 1950	h_o, ft Project Flood	h_o (max), ft Computed from i_e = 0.85	h_o (max), ft Observed	$\frac{h_o}{H}$ % 1950	Natural Seepage in gpm/100 ft of Levee (1950) Q_s	Q_s/H	Q_A**	Q_A/H**
C'ville[a]	26/0+00	A	9.4	19.2	1,200	330	-	100	1,500	Cl-Sl	6.5	17	Sl Si	5	90	230	560	240	240	2.0	3.2	3.2	2.0	21	260	28.0	-----	-----
Gammon	138/26+00	C	11.9	25.2	20,000	500	300	130	1,000	Clay	7.5	5	Clay	2	200	480	980	700	700	3.3	3.3	6.4	3.3	28	135	11.3	71.4	6.0
Commerce	23/10+75	B	9.2	22.7	1,800	400	1,400	165	1,000	Cl-Sl Si	7.0	2.0	Si Sl	3	500	1,100	1,500	1,000	950	2.2	3.2	3.2	2.2	25	91	9.9	60.7	6.6
Trot 51[b]	52/22+00	K	9.0	23.7	3,100	450	1,300	100	1,000	Cl-Si Sl	5.0	1.0	Si Sl	1	1,000	650	2,100	735	735	3.0	6.0	6.0	3.0	33	72.5	8.1	-----	-----
Trot 54[b]	54/1+05	M	13.8	27.7	2,500	475	-	90	1,250	Clay	9.0	1.4	Clay	1	730	575	1,050	770	770	3.0	3.0	7.7	3.0	22	125	9.1	55.2	4.0
Stovall	77/38+00	B	14.9	29.8	3,000	600	900	40	2,500	Sl-Cl	15.0	4.0	Sand	----	700	200	800	800	800	6.0	9.8	9.8	6.0	44	150	10.0	120.7	8.1
Yarrell	81/24+00	A†	6.8	24.1	5,000	500	-	20	300	Clay	4.0	1.22	Sl Si	2	245	80	580	140	140	2.4	2.4	2.4	2.4	28	8.2	1.2	-----	-----
		A††	6.8	24.1	5,000	500	-	70	1,000	Cl & Si	6.0	0.49	Sl Si	2	2,050	360	860	930	1,050	6.2	17.5	20.0	6.2	72	37.5	5.5	-----	-----
U Fran[c]	39+00	B	8.3	25.0	6,800	450	3,500	125	1,400		18.0	1.6	Clay	8	870	1,070	2,520	1,400	2,150	1.5	15.3	15.3	1.5	21	73	8.8	-----	-----
L Fran[c]	130+00	C	13.6	28.6	1,100	575	850	135	1,600	Clay	5.0	17	Sand	----	95	435	1,010	250	250	1.7	3.8	3.8	1.7	13	340	25.2	-----	-----
Pulivar	2199+75	D	6.5	26.2	1,500	330	900	50	1,200	Clay	13.0	10	Sl Si	5	115	320	650	365	365	2.4	4.2	11.0	2.4	37	101	15.6	-----	-----
Butaw	2060+00	D	6.2	26.1	2,500	450	-	70	1,100	Si Sl	18.0	1.3	Si Sl	6	875	1,150	1,600	1,050	1,050	6.0	11.0	11.0	6.0	65	27	4.3	-----	-----
L'Argent	3542+33	B	16.4	31.0	2,500	380	1,340	120	400	Clay	15.5	0.24	Clay	15	1,700	2,770	3,150‡	3,000	5,800	5.4	14.9	14.9	5.4	35	18	1.1	-----	-----
H W[d]	3618+95	D	10.4	24.5	2,100	500	-	130	500	Cl-Si Sl	4.0	0.72	Sl Si	3	690	1,600	2,100	600	620	1.2	1.9	1.9	1.2	13	40	3.5	-----	-----
Kelson	2700+71	B	16.7	21.7	1,000	180	400	6‡‡	5‡‡	Sl Cl	2.0	0.025	Clay§	15§	200‡‡	70	250	50‡‡	50	4.6	8.9	18.7	4.6	28	0.25	0.015	-----	-----
Baton R[e]	97+42	C	17.4	23.0	500	210	5,000	175	500	Sl Cl	30.0	0.06	Clay	20	9,000	590	800‡	11,200	11,200	12.4	15.2	26.8	12.4	73	18.7	1.1	9.05	0.52
Cotton[f]	826+49	B	4.5	17.0§§	1,050	200	2,300	30	300	Clay	7.5	------	Si Sl	10	-----	-----	-----	-----	-----	----	----	----	----	--	-----	-----	-----	-----

[a] Caruthersville.
[b] Trotters 51 and Trotters 54.
[c] Upper Francis and Lower Francis.
[d] Hole-in-the-Wall.
[e] Baton Rouge.
[f] Cotton Bayou.
* Soil types and thickness are approximate; both may vary considerably along the piezometer line.
** Q_A and Q_A/H are measured seepage flows in limited areas landward of levee and do not represent the total seepage passing beneath the levee.
† Upper aquifer.
†† Lower aquifer.
‡ Near high bank of Mississippi River.
‡‡ Values for natural levee deposit in which piezometers were installed.
§ Type and thickness of blanket over deep pervious sand; shallow natural levee stratum of silt in which piezometers were installed is exposed in riverside borrow pit.
§§ Interim flood.

particularly severe seepage between a landward swale closely paralleling a levee are shown at mile 52 of the Trotters 51 site (plate 61) and at Stovall (plate 103) during the 1937 high water. The wide swale landward of the levee at Stovall is also a good example of how such a swale can increase the excess head at the levee toe.

613. Higher elevation of the surface of point bar deposits landward of low ground can, in effect, also prevent the exit of seepage landward of the low topography because the substratum pressure may not be as high as the ground surface. A good example of this condition exists at the L'Argent site. In 1950 practically all seepage at line B emerged through the comparatively thick clay top stratum of a filled channel because the surface of bar deposits landward of the filled channel was high enough to prevent the emergence of seepage. Another example exists at the Eutaw site.

614. Clay plugs or channel fillings. Several piezometer sites were located in reaches where the top stratum consists of wide channel fillings (Upper Francis and L'Argent). At these sites, fairly high excess heads (h_o/H) developed landward of the levee during 1950; seepage emerging through the top stratum was apparently rather uniform and fairly light for the maximum river stage that developed. However, two sand boils developed through the channel fill at L'Argent in 1937 when the river stage was higher. At the Trotters 54 site, also located on a relatively uniform filled channel, high excess pressures did not develop because of relief provided by sand boils in a drainage ditch a short distance landward of the levee.

615. The occurrence of sand boils between a levee and clay-filled channels was illustrated at the Gammon and Lower Francis sites during the 1950 high water. However, the occurrence and location of sand boils at these sites are attributed more to the thinness of the top stratum landward of the levee and the close source of seepage than to the effect of the clay-filled channels.

616. Natural levee deposits. Several levees studied are founded on relatively continuous silty natural levee deposits underlain by clay (e.g., Upper Francis, Eutaw, and Kelson). At these sites, some minor

seepage probably occurs through the natural levee deposits, although to date no sand boils have been attributed to such seepage.

617. In general, it is believed that if a levee has adequate base width as compared to the net head, seepage through natural levee deposits will probably not be of a serious nature.

618. Backswamp deposits. The thickness of backswamp clay deposits usually precludes the development of serious seepage. However, if seepage has a ready entry into the pervious substratum, high substratum pressures can be expected to develop; and if, for any reason, the continuity of the clay is broken or the thickness of the clay is not adequate to withstand the uplift pressure, underseepage and possibly severe sand boils may develop. Although the top stratum at Baton Rouge is comprised of point bar deposits, the thickness is so great (30 ft) that the seepage pattern is similar to that expected along levees founded on thick backswamp clays; i.e., high excess heads and low rates of natural seepage.

Man-made features and seepage

619. Man-made discontinuities in the top stratum also influence the development of sand boils. Such discontinuities encountered at the sites studied consist primarily of landside drainage ditches and seismic shot points; however, they could also include cisterns and wells, fence-post holes, improperly backfilled borings, etc. Both riverside and landside borrow pits near a levee can have a pronounced effect on seepage. The effect of riverside borrow pits on underseepage is discussed later in this part.

620. Landside drainage ditches. The development of sand boils in landside drainage ditches is significant at the Gammon and Trotters 54 sites. During the 1950 high water at Gammon, numerous boils occurred in such a ditch which was only about 2 ft deep; practically no seepage emerged in the area landward of the ditch (plate 17). At Trotters 54, numerous boils and concentrated seepage occurred along a ditch about 2 to 3 ft deep during periods when the relief well system was not operating (plate 84). At both of these sites, the seepage would probably have been relatively uniform had it not been for the presence of the landside drainage ditches.

621. Seismic shot points. The occurrence of sand boils during the 1937 high water at the Baton Rouge site, where the top stratum generally consists of about 30 ft of clay and silt, is attributed to the weaknesses in the top stratum caused by seismic shot holes. The influence of the shot points or other similar discontinuities on sand boils at Baton Rouge is greatly magnified because unusually high landside substratum pressures develop.

622. Landside borrow pits. Landside borrow pits used for the construction of sublevees exist at Trotters 51 and Bolivar. These particular pits are not very deep and because of the relatively low river stages during the 1950 high water had little effect on seepage. However, because of the reduced thickness of the top stratum in the sublevee basins, the maximum safe excess head allowable is considerably less than would otherwise be permissible.

Characteristics of Riverside Top Strata

Source of seepage and effective length of riverside blanket

623. Of the 15 sites where sufficient piezometric data were available for analysis, the source of seepage at the crest of the 1950 high water was located in the riverside borrow pits at all sites except those where the borrow pits are blanketed with a thick layer of clay. Values of s ranged from about 600 ft to 3000 ft except at Kelson where s was only 250 ft for the silty aquifer. The corresponding effective lengths of riverside blanket x_1 ranged from about 200 ft to 2800 ft, excluding that at Kelson.

624. The effect of the type and thickness of blanket materials in riverside borrow pits on s and x_1 is shown in table 36. At L'Argent and Baton Rouge the riverside borrow pits contain a blanket of clay about 15 and 20 ft thick, respectively, and the effective source of seepage was located at about the bank of the Mississippi River. At the other sites the thickness of the blanket in the borrow pits ranged from about 0 to 8 ft, and the source of seepage was generally in the riverside borrow pits.

At sites where either a thick blanket existed in the borrow pits (L'Argent and Baton Rouge) or where the pervious substratum was exposed in the borrow pits (Stovall and Lower Francis), little or no change in s occurred with rising river stages. However, at sites where a thin blanket was present in the borrow pits, a decrease in s and x_1 resulted during rising river stages (e.g., Trotters 51, and line R at Trotters 54). The decrease in s and x_1 may possibly be attributed to some scour in the borrow pits during the high water, which tends to reduce the effective thickness of the blanket material in the pits. Effective lengths of riverside blankets x_1 for various types and thicknesses of blanket in riverside borrow pits as determined from boring and piezometer data are summarized in table 37.

625. Values of x_1 in table 37 are observed values adjusted to an assumed condition of a riverside blanket of infinite length with the same thickness as that in the borrow pit. This adjustment was made by means of blanket formulas given in Part III and was intended to partially eliminate the effect of different top strata riverward of the borrow pits and different distances between the levee and river at the various sites.

Table 37

Summary of Distances to Effective Source of Seepage, Effective Lengths of Riverside Blankets and Vertical Permeability of Riverside Blanket Materials at the Crest of 1950 High Water

Blanket in Riverside Borrow Pit		Number of Piezometer Lines from Which Data Were Obtained	s, ft			x_1, ft*			$k_{bR} \times 10^{-4}$ cm/sec			Suggested Design Values	
Soil Type	Thickness in ft		Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	k_{bR}	x_1
Sand	---	3	1080	800	960	480	200	370	----	----	----	----	250
Silty Sand**	<5	3	800	560	670	320	230	280	14.2	1.6	7.0	7.0	300
	5 to 10	1	560	560	560	280	280	280	1.8	1.8	1.8	2.5	600
											5.7†		
Silt & sandy silt	<5	4	1500	600	1050	1220	270	670	7.4	0.24	2.2	2.0	400
	5 to 10	2	1600	910	1260	1190	510	850	5.0	0.33	2.7	1.5	800
	>10										2.4†	1.0	1200
Clay	<5	6	1280	610	1020	750	110	690	1.7	0.34	0.79	0.8	600
	5 to 10	2	1720	1520	1620	1270	1070	2170	1.3††	0.86††	1.08††	0.5	1300
	10 to 15	0	----	----	----	----	----	----	----	----	----	0.2	2500
	>15	3	3150	800	1600	∞	∞	∞	0.0	0.00	0.00	0.05	4000 or L_1‡
											0.4†,‡‡		

* Values of x_1 computed from observed values of x_1 and adjusted to a condition where $L_1 = \infty$.

** Does not include Hole-in-the-Wall where values of s and x_1 may not be reliable because artesian flow conditions did not develop until near the crest of the 1950 high water.

† Averages of all values of k_{bR} for a given soil type without regard to thickness.

†† Values are considered to be too high as at these piezometer lines (Upper Francis) seepage could enter the pervious substratum through a silty blanket riverward of the borrow pit as well as through the clay in the borrow pit.

‡ Use the smaller of the two values.

‡‡ Average does not include k_{bR} for blanket thickness between 5 and 10 feet.

At most sites the distance from the levee to the river was far enough to have little effect on the value of x_1 or s. It is pointed out that the ty and thickness of blanket in riverside borrow pits may vary considerably at any one site and, therefore, the soil types and thicknesses of blankets given in tables 36 and 37 are only general or approximate values as are the corresponding values of k_{bR}. Scatter in the data can also be expected because of the limited number of piezometer lines falling into each of the categories in table 37.

626. At L'Argent and Baton Rouge, x_1, observed, was about 2770 and 620 ft, respectively; these distances correspond approximately to the respective distances L_1 between the riverside toe of the levee and the river.

627. From a cursory examination it may appear that the assumption that the blanket in the riverside borrow pit is infinite rather than finite in riverward extent, with a different thickness of top stratum between the borrow pit and river, may not be valid. However, except at L'Argent and Baton Rouge, the values of x_1 for $L_1 = \infty$ were practically identical with the observed values of x_1 for a finite L_1. This close agreement indicates that at sites where a thin blanket remains in the riverside borrow pit and the pit has a width of about 300 to 1000 ft, as at the sites studied, the natural top stratum riverward of the borrow pit has little effect on the effectiveness of the riverside blanket and resulting seepage pattern. This phenomenon is further verified from consideration of blanket formulas, which show that for a uniform riverside top stratum infinite in riverward extent 65% of the total quantity of seepage enters the pervious substratum through the riverside blanket within a distance of x_1 from the levee. Exceptions to the above will occur at sites where the blanket in the borrow pit is fairly impervious and abc equal in thickness to that riverward of the borrow pit (L'Argent and Baton Rouge).

628. From table 37 it appears that x_1 generally tends to increase as the blanket material in the borrow pits grades from silty sand to clay and also as a given type blanket increases in thickness. It can be seen that the silty sand blankets were not very effective, as the x_1 values

are about the same as those at sites where the substratum sands are exposed in the borrow pits. Where a clay blanket with a thickness of 15 ft or more occurs, values of x_1 are about the same as L_1, indicating that for practical purposes little seepage can be expected to penetrate such a blanket.

629. The values of s obtained for different types and thicknesses of blankets are also shown in table 37, and although there is a tendency for s to increase as the top stratum thickness increases and the blanket grades from silty sand to clay, the trend is not as pronounced as it was for x_1. This is because values of s include the base width of levee and berm (L_2), which varies for the sites, as well as x_1.

630. The accuracy of s and x_1 depends to a considerable extent on the accuracy of readings of the piezometers perpendicular to and beneath the levee. Water levels in piezometers at the sites studied were recorded to the nearest 0.1 ft. Assuming a riverside piezometer reading to be 0.05 ft too low and a landward piezometer reading to be 0.05 ft too high, the observed value of s would be about 3 to 15% in error for the range of s values observed at the sites. Similarly, the values of x_1, either observed or adjusted, may be in error as much as 10 to 20%. These possible variations probably account for some of the scatter of s values.

631. It should also be noted that s (and x_3) at several sites was computed from readings of two shallow piezometers: one at the landside toe of the levee or berm and one beneath the levee. At such sites, prior to computing s and x_3, it was necessary to estimate the average head in the substratum sand at the levee toe from the reading of the shallow piezometer at the toe and from fig. 26. If the actual pressure at the middepth of the sandy substratum was ± 0.1 ft from that assumed, the true value of s could be 3 to 10% different from the computed value. It is possible that at some sites the applied head correction may deviate from the actual head difference by more than 0.1 ft; such deviation would also contribute to the scatter occurring in table 37. At new piezometer installations, two shallow piezometers should be installed <u>beneath</u> the levee (one riverward and one landward of the crown) -- or one shallow piezometer beneath the levee and one piezometer at the middepth of the

pervious substratum at the toe of the levee -- to eliminate the need for applying a head correction with resultant uncertainties in the value of s .

632. At sites along Mississippi River levees where values of s must be estimated without the benefit of piezometric data, it is believed that reasonable estimates can be obtained from table 37 by selecting the proper value of x_1 on the basis of the riverside blanket, and then adding x_1 to the base width of levee (and berm, if present). This procedure should give fairly reliable values of s at sites where the width of the riverside borrow pit exceeds 300 ft and where L_1 is at least 1500 ft.

633. At sites where the borrow pits are narrower than 300 ft, the values of x_1 may be slightly greater than those in table 37; where the levee is closer to the river than 1500 ft, x_1 may be less than indicated in table 37. Estimation of x_1 and, hence, s , at sites where the width of borrow pit is less than 300 ft or where L_1 is less than 1500 ft can be made from blanket formulas and a knowledge of the permeability of the blanket material k_{bR} . Values of k_{bR} are discussed below.

Permeability of riverside blanket

634. The vertical permeability (k_{bR}) of the riverside blanket was determined at each site using equation 5 and the average thickness (z_{bR}) of the blanket, the effective length of riverside blanket (x_1 for $L_1 = \infty$) at the crest of the 1950 high water, k_f , and d . Values of k_{bR} for various types and ranges in thickness of blanket are summarized in table 37.

635. It appears that, for a given material, k_{bR} generally tends to decrease as the blanket thickness increases, particularly for clay blankets. Values of k_{bR} were zero at sites where z_{bR} equalled or exceeded 15 ft of clay as compared to about 1.0×10^{-4} cm per sec where the clay blanket was less than 5 ft thick. No apparent decrease in k_{bR} with increasing blanket thickness was observed at sites where the borrow pits were blanketed with silt (less than 10 ft thick). The average permeability of silty blankets was about 2.5×10^{-4} cm per sec; k_{bR} for

silty sand blankets with thicknesses up to 10 ft averaged about 6×10^{-4} cm per sec.

636. Although reasonable trends were obtained for k_{bR} as determined from adjusted values of x_1 at the crest of the 1950 high water, it is pointed out that the values of z_{bR} were obtained from riverside borings and cross sections of the borrow pits made prior to the 1950 high water. If significant changes in the thickness of the blanket in the borrow pits occurred in the interval between the date of the survey and the crest of the high water, as a result of either deposition or scour, the corresponding values of z_{bR} and k_{bR} would differ somewhat from the computed values.

637. Even though the suggested design values of k_{bR} shown in the last column of table 37 may be no more than $\pm 50\%$ accurate, in the absence of actual piezometer data they are considered more reliable than could be estimated or computed from laboratory test data.

Characteristics of Landside Top Strata

Effective seepage exit

638. Values of x_3 at the crest of the 1950 high water are shown in table 36. They generally ranged from about 150 to 11,000 ft with the largest x_3 occurring at Baton Rouge where the top stratum is about 30 ft thick. At Kelson, x_3 was only 50 ft. However, this value is not comparable to those at the other sites because it is related to a thin aquifer of low permeability (5 ft of silt), whereas at the other sites the values of x_3 correspond to conditions where the top stratum is underlain by thick, pervious sand substratum. The distance to the effective seepage exit was usually rather short at sites where the landside top stratum was thin (Lower Francis and Farrell) and at sites where numerous sand boils developed (Caruthersville and Gammon). At sites where the exit of seepage was partially blocked as a result of landward swales or sloughs (Gammon, Stovall, and Lower Francis), the observed values of x_3 and h_o were larger than if the block were not present.

639. The accuracy of x_3 also depends upon the accuracy of the

piezometer readings used in its determination. As discussed previously for s , values of x_3 may be in error as much as 5 to 20% as a result of inaccuracies in river stage and piezometer readings, and in the estimation of head at the middepth of the aquifer at the landside toe of the levee.

640. The determination of x_3 also depends upon the assumed elevation of the average ground surface or tailwater. For example, if the assumed average ground or tailwater elevation is 0.5 ft in error, a corresponding 10 to 20% error will exist in the value of x_3 . At sites where the tailwater elevation was recorded and the submerged area was close to the levee toe, the values of x_3 are considered reliable. However, where the area was not submerged and the ground was not level, the assumed average ground elevation is probably only accurate to within ±0.5 ft, and values of x_3 may be limited in accuracy. To reduce the possibility of utilizing incorrect tailwater assumptions at the 16 sites studied and at possible future similar installations, the tailwater elevations in submerged areas should be recorded at frequent intervals during high-water periods.

641. Computed values of x_3 are also affected by the rate at which ground-water storage is filled beneath and landward of the levee during a high water, because formulas for computation of x_3 are valid only for artesian flow conditions. At sites such as Lower Francis, where the piezometers rapidly reflected the effect of rising river stages, the values of x_3 are probably indicative of the seepage pattern throughout most of the high water. However, at sites such as Hole-in-the-Wall, where artesian flow conditions did not develop until near the crest of the 1950 high water, most of the computed values of x_3 are meaningless. In view of this, care must be exercised when analyzing piezometric data obtained during the initial portion of a high water, as false indications will result if artesian conditions do not exist.

642. In addition to the above limitations in accuracy of x_3 , comparisons between the values of x_3 given in table 36 can be misleading because the only similarity in these values is that they represent conditions that existed at the crest of the 1950 high water. At this

river stage, sand boils had developed at some sites, medium to light seepage at other sites, and ground-water storage was still filling at other sites. For purposes of comparison, values of x_3 are also given in table 36 for the river stage at which upward gradients through the landside top stratum had become or would become critical. These values of x_3 were obtained from the figures in Part IV showing x_3 plotted against river stage, and are believed to be fairly comparable, as by the time the upward gradient through the top stratum reaches the critical value, conditions of artesian flow will have developed. Furthermore, these values of x_3 are estimated to be about the greatest that will occur at each site based on piezometer readings at the toe of the levee (see paragraph 133).

643. A comparison between x_3 at the crest of the 1950 high water and that observed or estimated to exist when the upward gradient becomes a maximum indicates that at most sites little or no difference occurred. This is attributed to either the development of about the maximum possible upward gradient during the 1950 high water (Gammon, Trotters 54, and Lower Francis), or the estimation that x_3 would remain constant up to the crest of the project flood (Baton Rouge). The greatest difference in x_3 occurred at L'Argent and Upper Francis, where x_3 continued to increase with river stage at the crest of the 1950 high water. Although generally little or no difference occurred in most of the x_3 values as a result of projecting data to conditions where $i = i_c$, whenever significant changes occurred, they were considered in the estimation of k_f/k_{bL} as described later.

Thickness and permeability of landside top stratum

644. It can be seen from table 36 that a large variation occurs in type and thickness of top stratum materials at the 16 sites. Excluding the thin clay stratum at Kelson overlying the silty natural levee deposits, the thinnest landside top stratum encountered was about 4 ft thick (Farrell). The thickest top stratum encountered was 30 ft at Baton Rouge. A review of data from all piezometer lines analyzed, except those at Kelson and Cotton Bayou, shows that k_{bL} ranged from about 0.06×10^{-4}

cm per sec at Baton Rouge to 40 x 10^{-4} cm per sec at line D at Gammon. Comparatively high values of k_{bL} were found at sites where there was a tendency for large quantities of seepage to emerge landward of the levee (Caruthersville, Gammon, and Lower Francis). In considering the values of k_{bL} shown in Part IV and in table 36, it must be remembered that they were computed from piezometer data obtained at the crest of the 1950 high water and reflect various degrees of seepage, depending on the site. As x_3's at a site are known to vary with river stages (see graphs of x_3 vs H , Part IV), so does k_{bL} vary with river stage and seepage (particularly sand boils) conditions. Most values of k_{bL} at the crest of the 1950 high water ranged from about 0.5 to 10 x 10^{-4} cm per sec.

645. The relationship between k_{bL} and z_{bL} as obtained at the various piezometer sites is shown in fig. 45; also shown is the corresponding classification of the top stratum. Soils with composite

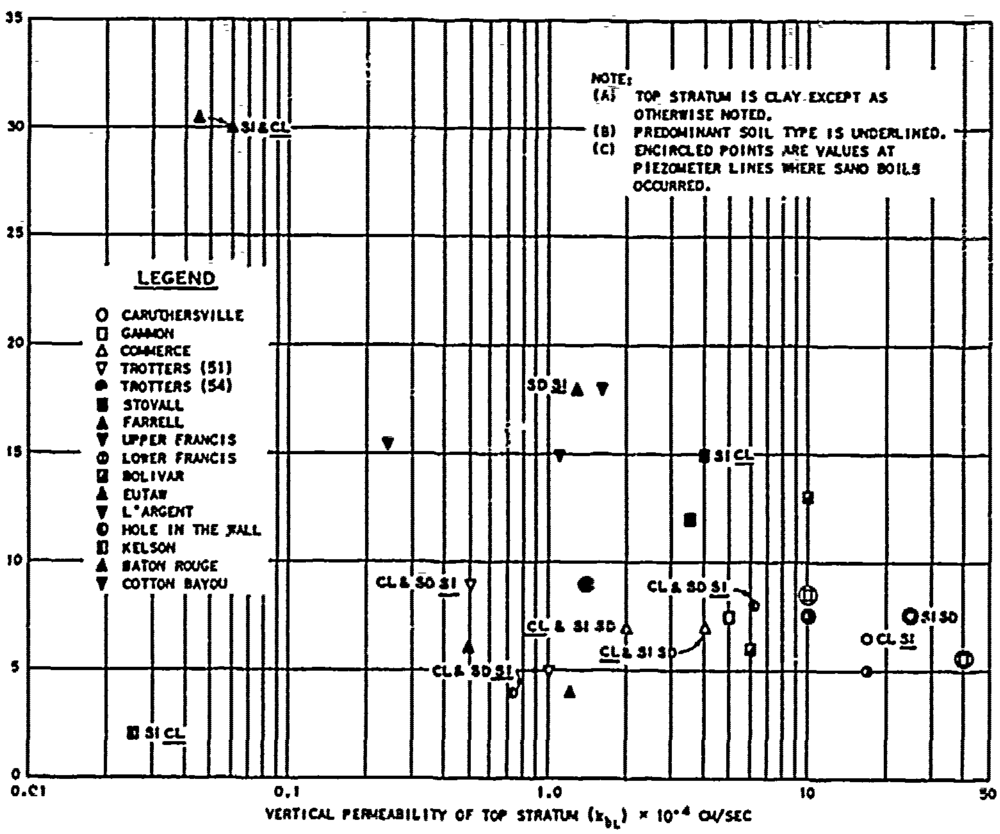


Fig. 45. Permeability of landside top stratum at crest of 1950 high water

classifications such as "clay and sandy silt" were classed as either clay or silt for purposes of correlation, depending on the predominant soil type. The predominant soil type is underlined in fig. 45.

646. Fig. 45 illustrates that there is a pronounced trend for k_{bL} to decrease with an increase in z_{bL}, particularly for clay-type top strata. Trends shown in fig. 45 are also shown in table 38 from which it is seen that, although k_{bL} decreased with increasing thickness of clay top strata, there was a lesser tendency for k_{bL} to decrease with increasing thickness of silt top strata. The permeability of top strata less than 10 ft thick was about the same for silt as for clay -- about 5×10^{-4} cm per sec. The permeability of the single silty sand blanket was 25×10^{-4} cm per sec.

647. It is apparent that the above values of k_{bL} are greater (perhaps 100 to 1000 times) than would normally be obtained from laboratory permeability tests on undisturbed samples of silts and clays in the Lower Mississippi Valley. Also, in the laboratory silts are usually more permeable than clays. The high values of k_{bL} obtained (fig. 45 and table 38) show that the permeability of top strata landward of levees is not related to the values obtained from laboratory tests on undisturbed samples, but instead to the presence and number of fissures, root holes, former boil holes, and other perforations in the top stratum. The effect of these perforations in clay top strata appears to be reduced if the blanket thickness exceeds 10 ft, and greatly reduced if z_{bL} exceeds 15 ft. This may also be true for silty blankets, but data for such blankets were not sufficient to establish a trend.

648. As discussed previously, values of k_{bL} were also computed for a condition where the upward gradient through the top stratum was observed or estimated to first become critical; the averages of these values are given in table 38. However, there is no significant difference between these values and those computed at the crest of the 1950 high water.

649. Comparisons between k_{bR} and k_{bL} for similar blankets of similar thickness (tables 37 and 38) indicate that the landside blanket

Table 38

Summary of Ratios of Permeability of Pervious Substratum to Landside Top Stratum and Permeability of Landside Top Stratum at Crest of 1950 High Water

Soil Type	Thickness in ft	Number of Piezometer Lines from Which Data Were Available	k_f/k_{bL} Max	k_f/k_{bL} Min	k_f/k_{bL} Avg*	$k_{bL} \times 10^{-4}$ cm/sec Max	$k_{bL} \times 10^{-4}$ cm/sec Min	$k_{bL} \times 10^{-4}$ cm/sec Avg**	Avg k_{bL} in 10^{-4} cm/sec from x_3 at $i = i_c$	Suggested Design Values $k_{bL} \times 10^{-4}$ cm/sec†	Suggested Design Values k_f/k_{bL}††
Silty sand	<5	0	-----	-----	------	-----	----	----	----	10	125
	5 to 10	1	-----	-----	60	-----	----	25	25	8	150
	>10	0	-----	-----	------	-----	----	----	----	6	200
Silt and sandy silt	<5	2	1,000	690	840	1.0	0.7	0.9 } 5.1	0.8 } 4.5	5	250
	5 to 10	3	2,000	81	445	17	0.5	7.9 }	6.3 }	4	300
	10 to 15	0	-----	-----	------	-----	----	----	----	3	400
	>15	1	-----	-----	875	-----	----	1.3	1.3	2	600
Clay and silty clay	<5	3	-----	-----	95‡	17	0.03	6.1 } 8.1	6.0 } 7.9	4	250
	5 to 10	9	2,050	25	345	40	0.5	8.8 }	8.5 }	3	400
	10 to 15	4	1,270	115	640	10	1.1	4.7	4.3	1.5	800
	15 to 20	3	1,700	870	1,130	1.6	0.24	1.0	----	0.5	2,500
	>20	2	9,000	8,350	8,600	0.06	0.06	0.06	----	0.08	15,000

* Average = square of average of square roots of individual k_f/k_{bL} values.

** Arithmetic average.

† Values based largely on observations at the piezometer sites. They may be somewhat high for new levees and levees which have not been subjected to major high water.

†† Based on $k_f = 1250 \times 10^{-4}$ cm per sec.

‡ Value of k_f/k_{bL} for one piezometer line only. Values of k_f/k_{bL} at other two lines are not indicative of the ratio of the permeability of the entire effective pervious substratum to that of the landside top stratum because the tips of the piezometers are installed in a silty sand or fine sand stratum which is separated from the principal seepage carrying aquifer by clay strata.

tends to be about 2 to 10 times as pervious as the riverside blanket. As cracks and fissures probably exist on both sides of the levee, this difference is attributed to the tendency of upward seepage through the landside top stratum to flush existing cracks and perforations, thereby increasing the over-all permeability of the top stratum. Downward seepage through the riverside blanket tends to seal any cracks and fissures unless excessive erosion occurs.

650. In the design of control measures at sites where piezometric and seepage data are not available, values of k_{bL} can be estimated from the average values given in table 38. However, it should be noted that these values are for conditions at the crest of a fairly high water; therefore, to insure that seepage conditions do not become as severe, control measures should be designed on the basis of values of k_{bL} somewhat less than the average values given in table 38. Suggested values of k_{bL} for design purposes are also shown in table 38.

Characteristics of Pervious Substratum

651. The effective thickness d and permeability k_f of the pervious substratum at the sites are given in table 36. The effective thickness of the pervious aquifer at the various piezometer sites was taken as the depth of sand below the upper strata of clay, silt, and very fine or fine sands ($k < 200 \times 10^{-4}$ cm per sec). The permeability k_f is the average horizontal permeability of all strata included within the effective thickness d .

652. In general, the effective thickness of the pervious substratum ranged from about 70 to 165 ft and averaged about 110 ft for the sites along the Mississippi River, except at Stovall where d is only about 40 ft. At Cotton Bayou on the Red River, d was only about 30 ft.

653. Estimated values of k_f ranged from about 400 to 1600 x 10^{-4} cm per sec at the sites along the Mississippi River except at Stovall where k_f may be as high as 2500 x 10^{-4} cm per sec. At Cotton Bayou k_f was estimated to be only about 200 x 10^{-4} cm per sec.

654. For sites in the Memphis District and in the Vicksburg District

above L'Argent, k_f in general ranged from about 1000 to 1500 x 10^{-4} cm per sec; at L'Argent and piezometer sites farther downstream, k_f was estimated to be no greater than about 500 x 10^{-4} cm per sec. From recent pumping tests performed at the sites of the Old River control and lock structures, located about 45 miles south of Natchez, Miss., on the right bank of the Mississippi River, k_f values of 1000 and 600 x 10^{-4} cm per sec, respectively, were obtained. Thus it should not be inferred that k_f will always be less than 500 x 10^{-4} cm per sec in the alluvial valley below L'Argent; however, lower values of k_f generally can be expected below L'Argent than farther upstream.

655. A detailed summary of permeability values of the pervious substratum, as estimated for each site by various methods, is given in table 39. At Commerce and Trotters 54 where the coefficient of permeability of the foundation was well defined from analyses of piezometric data and natural seepage measurements, well-flow data, and pumping tests, the values of k_f obtained by means of grain-size data and fig. 17 were within 5% of the average value of k_f obtained from the above three determinations. At Trotters 51 where permeabilities were obtained from piezometric, seepage, and well-flow data, the value of k_f obtained from grain-size data and fig. 17 was within 1% of the average of the above two determinations. In view of these close agreements, values of k_f obtained from grain-size data and fig. 17 were included when determining k_f at these three sites.

656. Poor agreement was obtained between the values of the permeability of the substratum estimated from laboratory permeability tests and those values obtained from piezometric, seepage, well-flow data, and pumping tests at the three sites mentioned above. This is attributed largely to the fact that the permeability of remolded samples of most sands is not as great as the horizontal permeability of sands in situ. The poorest agreement was at Trotters 54. At this site the laboratory permeability data were obtained from boring M-56 which was advanced by use of drilling mud; it is possible the samples were contaminated by the mud, which would greatly reduce the permeability as determined in the laboratory. In general, the use of laboratory permeability data to estimate k_f at the

Table 39

Summary of Coefficient of Permeability (10^{-4} cm/sec) of Pervious Substratum Obtained by Various Methods

Site	Laboratory Permeability Tests	Grain-size Data and Fig. 17	Seepage and Piezometric Data	Field Pumping Tests	Well Flow Data	Selected Value of k_f
Caruthersville	1150	----	----	----	----	1500
Gammon	750	1200	850	----	----	1000
Commerce	750	900	875*	1000	865	1000
Trotters 51	750	1000	835	----	1150	1000
Trotters 54	400	1250	1250	1180	1500**	1250
Stovall	350	950	2650†	----	----	2500
Farrell††	800	1200	----	----	----	1000
Upper Francis	900	1900	----	----	----	1400
Lower Francis	1100	2300‡	----	----	----	1600
Bolivar	----	1310	----	----	----	1200
Eutaw	----	1310	----	----	----	1100
L'Argent	----	350	----	----	----	400
Hole-in-the-Wall	60	500	----	----	----	500
Kelson‡‡	0.6	----	----	----	----	5
Baton Rouge	----	----	600 §	----	----	500
Cotton Bayou	30	200	----	----	----	200

* Piezometer Line H.
** 1951 data.
† Average for piezometer lines A and B.
†† Values are for lower aquifer only.
‡ Piezometer Line C.
‡‡ Values are for upper stratum of silty sand.
§ Piezometer Line A.

sites studied gave values somewhat lower than those determined from pumping tests or estimated from grain-size data and fig. 17. The selected values of k_f for the various piezometer sites were based largely on the following: pumping tests or well-flow and piezometer data, where available; the relation between k_f and D_{10} (as shown by fig. 17); partially on seepage and piezometric data, where available; and to a very limited extent on laboratory permeability test data. At Kelson, where the pervious stratum investigated consisted of silty sand, k_f was estimated from laboratory permeability tests and judgment, as fig. 17 does not apply to finer grained soils. At Caruthersville and Baton Rouge it was not

possible to estimate k_f from grain-size data, as such data were not available. Although pumping test, well-flow, and seepage data were not available for all of the piezometer sites, it is believed the values of k_f given in tables 36 and 39 are reasonable estimates of the horizontal permeability of the substratum sands.

657. From the comparisons made in paragraph 655 it is believed that at sites along the Middle and Lower Mississippi River Valley where natural seepage, pumping test, or well-flow data are not available, k_f can be estimated reasonably well from grain-size data and fig. 17. However, if an accurate value of k_f is required, field pumping tests should be performed.

Ratio of Permeability of Pervious Substratum to Landside Top Stratum

658. Values of k_f/k_{bL} obtained at the crest of the 1950 high water at typical piezometer lines are given in table 36 for each site. These values range from about 100 to 2000 except at Baton Rouge where $k_f/k_{bL} = 8500$. A review of all values of k_f/k_{bL} reveals that values as low as 25 and 60 were obtained. The values of k_f/k_{bL} for the lower aquifer at Farrell are probably not comparable to those at other sites in view of the unusual characteristics of the top stratum for the lower aquifer at piezometer line A (see plate 120).

659. The relationship between k_f/k_{bL} and z_{bL}, x_3, and d is shown in fig. 46, from which it may be seen that there is a tendency for k_f/k_{bL} to increase as the thickness of clay top stratum increases. At sites where fairly thick clay top strata are present (L'Argent and Baton Rouge), k_f/k_{bL} was quite high. This trend becomes more apparent from the summary in table 38, which relates values of k_f/k_{bL} to thickness and type of top stratum. The average values of k_f/k_{bL} in table 38 were obtained from the square of the average of the square roots of the individual permeability ratios, because k_f/k_{bL} was based to a large extent on and varied as x_3 squared.

660. As shown in table 38, average values of k_f/k_{bL} increased

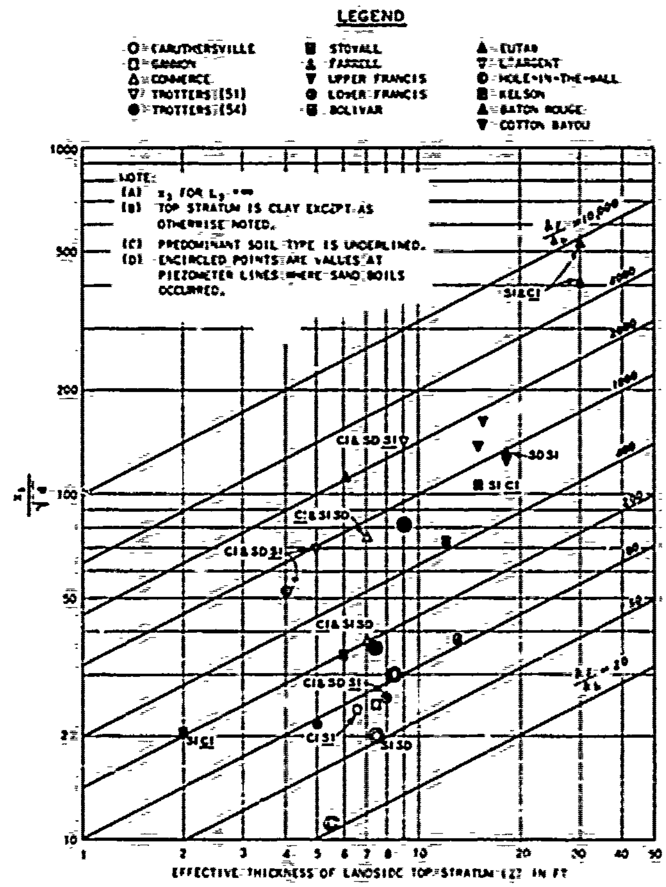


Fig. 46. Ratio of permeability of pervious substratum to permeability of landside top stratum at crest of 1950 high water

from about 100 to 8600 as z_{bL} increased from less than 5 ft to >20 ft at sites with clay top stratum. This increase is attributed to the fact that k_{bL} tends to decrease with increasing z_{bL} for clays, as previously discussed, and apparently predominated over variations in k_f in the over-all values of k_f/k_{bL}.

661. The average permeability ratio for sites where the top stratum is predominantly silty ranged from 445 to 875, with an over-all average of 630. This over-all average is about equal to that for sites with clay top stratum having a thickness of 10 to 15 ft. No apparent increase in k_f/k_{bL} with increasing z_{bL} was detected, and for top stratum thicknesses of 10 ft or less k_f/k_{bL} was greater at silty sites than at clayey sites.

662. Only one piezometer line was installed beneath a silty sand

top stratum (line C, Caruthersville) where a permeability ratio of 60 was obtained.

663. Although suggested values of k_f/k_{bL} based on the data in table 38 are given for designing seepage control measures at sites where piezometric data are not available, it is considered preferable to estimate k_{bL} from data in table 38, k_f from grain-size data and fig. 17 or field pumping tests, and then compute the permeability ratio.

Critical Upward Gradient

664. Maximum upward gradients through the top stratum observed in 1950, as measured by piezometers at the various sites, and the degree of seepage at or near each piezometer are given in summary tables for each site in Part IV. These upward gradients have been plotted against the corresponding seepage condition in fig. 47. From this figure the following general trends have been noted.

Seepage Condition	i
Light to no seepage	0 to 0.5
Medium seepage	0.2 to 0.6
Heavy seepage	0.4 to 0.7
Sand boils	0.5 to 0.8

665. It should be noted that the gradient required to cause sand boils varies considerably at the different sites, and relatively low gradients were recorded near some sand boil areas. This may be due to the fact that at sites where sand boils developed previous to the 1950 high water, only fairly low excess heads may have been required to reactivate boils in 1950 and, as a relief of pressure occurs at the boil, readings of piezometers near the boil may be somewhat lower than those farther from the boil. At sites where sand boils have not occurred in the past, higher gradients may have been required to initiate formation and development of sand boils, although this is difficult to ascertain because of limited past seepage data at several sites.

666. From the above data it appears that a seepage problem should be anticipated whenever estimated upward gradients exceed 0.5 to 0.8. Thus, it is generally recommended that seepage control measures be

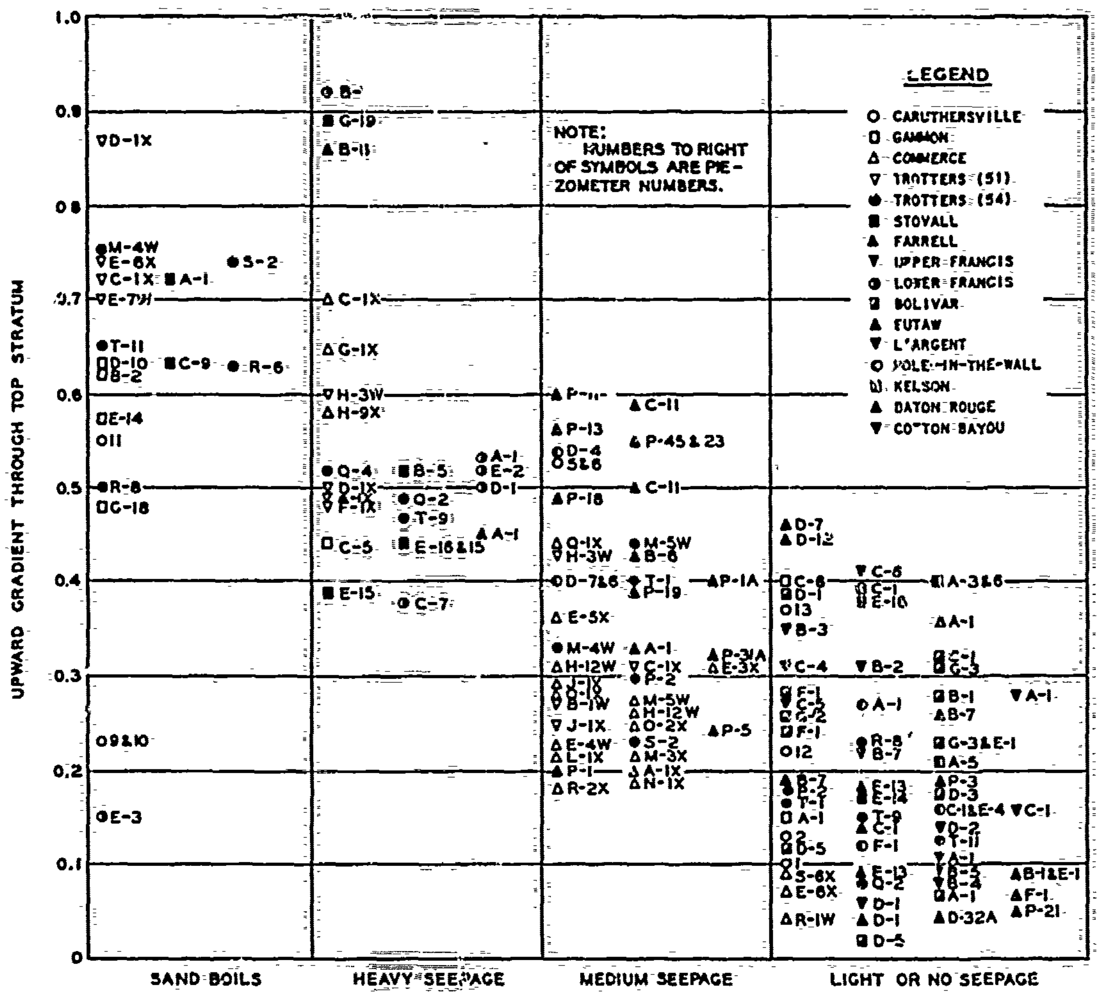


Fig. 47. Severity of seepage as related to upward gradient through top stratum

provided whenever seepage or piezometer observations indicate that critical uplift pressures will develop at river stages equal to or less than project flood stages. If such observations are not available, the uplift gradient should be estimated; and if it is greater than 0.75, seepage control is recommended. Methods of seepage control and their design are discussed in Part VI.

Effect of Natural Partial Cutoffs and Massive Clay Deposits on Seepage

Natural cutoffs

667. Piezometer lines were installed across natural partial cutoffs

at Stovall and Eutaw to measure the drop in head across such cutoffs. An examination of the piezometric gradients for these locations at the crest of the 1950 high water (plates 112 and 177) shows no significant drop in head across the partial cutoffs. This confirms analyses and model studies of partial cutoffs described in Part VI and in Appendix B.

Massive clay deposits

668. Massive clay deposits lie landward of the levee at certain piezometer lines at Gammon, Trotters 51, Stovall, Lower Francis, and Bolivar. The top stratum landward of the levee at Gammon and Lower Francis is rather thin, and the thick clay deposits landward of the levee probably had relatively little effect on the substratum pressures and seepage at these sites, except where the thick clays intersect the levee. However, it is believed that the clay deposits immediately landward of the levee at Trotters 51 and Stovall had a very significant bearing on the severe seepage conditions that occurred during both the 1937 and 1950 high waters. It is not possible to make a numerical comparison between pressures and seepage at these sites with and without such clay deposits except by means of theoretical blanket formulas. The clay swale landward of the levee at Stovall is estimated by blanket formulas to have increased the head at the levee toe at piezometer line A about 80% above that which would have existed at the crest of the 1950 high water had the swale not been present.

Efficacy of Seepage Berms at Piezometer Sites for Controlling Underseepage

669. Except for the seepage berm at Gammon, the berms at the other piezometer sites are of such soil type and/or thickness as to make them practically impervious. (Although some of the berms are as, or more, pervious than the underlying top stratum, they are thick enough to prevent emergence of seepage at the surface of the berm, and they do not have adequate hydraulic carrying capacity to carry any appreciable quantity of seepage landward through the berm. Therefore, they behave essentially as a practically impervious berm.) Thus, assuming that the

riverside and landside blankets remained unchanged after construction of the berms, these berms reduced seepage and landward pressures only in proportion to the amount their width increased the value of $x_1 + L_2 + x_3$ where L_2 is the original base width of the levee. As the original value of $x_1 + L_2 + x_3$ probably averaged about 1500 ft, the addition of a 200-ft-wide berm (typical of most sites) probably decreased seepage and landward pressures by approximately 10 to 15% from those that would have occurred without any berm. Since borrow for most of these berms was obtained riverward of the levee, the borrow operations may have reduced the effective value of x_1 as much as the width of the berm increased L_2. If such is the case, no reduction in Q_s or h_o would result from building the berm.

670. Construction of the rather thick berms at certain of the piezometer sites has practically eliminated the possible occurrence of sand boils at the landside toe of the levee. Such berms have also lengthened the path of any potential piping channel that might endanger the levee. However, as illustrated by the large sand boil 200 ft from the levee at Stovall in 1937, a 100- or 200-ft-wide berm does not in itself assure complete safety against underseepage. Nevertheless, a seepage berm does protect the landside toe of a levee against sloughing as a result of saturation from either through seepage or underseepage and will, if properly designed, adequately control underseepage.

671. The need for adequate exploration landward of a levee before designing and constructing a berm is well illustrated by the Trotters 51 and Stovall sites and, to a certain extent, by the site at Commerce. Because of the existence of rather massive clay deposits a short distance landward of the levee, the construction of impervious berms at these sites reduced the area in which seepage might naturally emerge. Such a concentrating effect tends to increase seepage between the landside toe of the berm and the massive clay deposit.

PART VI: DESIGN OF UNDERSEEPAGE CONTROL MEASURES

Control Measures and Criteria for Design

672. The control of underseepage and prevention of sand boils landward of levees founded on deep strata of pervious sands require some measure that will control erosional seepage and reduce excess pressure beneath the landside top stratum to a safe value.

673. Methods that may be used to control seepage are impervious riverside blankets, relief wells, landside berms, drainage blankets, drainage trenches, cutoffs, and sublevees. The choice of a control measure depends upon a number of factors, including the character of the foundation, cost, permanency, availability of right of way, maintenance, and disposal of seepage water. The principles involved in each of these methods of control are quite different. Where the pervious substratum is exposed riverward of a levee, an impervious riverside blanket acts to control seepage by increasing the resistance to seepage entry into the pervious substratum, thereby decreasing both seepage flow and excess pressure landward of the levee. An impervious cutoff beneath a levee blocks the passage of seepage beneath the levee even though there is a ready entry for seepage into the pervious foundation through the river channel or riverside borrow pits. Instead of blocking the flow of seepage beneath a levee, relief wells along the landside toe of a levee provide pressure relief and controlled seepage outlets that offer little resistance to flow but at the same time prevent erosion of the soil. A landside berm controls underseepage by increasing the thickness of the top stratum immediately landward of the levee so that the combined weight of the berm and top stratum is adequate to resist the excess uplift pressure, and by increasing the path of seepage flow through the pervious aquifer to the extent that the residual excess pressure at the toe of the berm is no longer critical. Filling sublevee basins with water reduces the activity and danger of sand boils by counterbalancing the excess head beneath the top stratum in the area encompassed by the sublevee. Drainage blankets and drainage trenches control seepage by intercepting it as it

emerges from the pervious substratum without allowing erosion to take place. They also provide a certain amount of pressure reduction landward of levees where the blanket or trench contacts the underlying aquifer.

674. For reasons subsequently discussed, only riverside blankets, relief wells, and seepage berms are generally recommended for the control of seepage beneath levees along the middle and lower reaches of the Mississippi River.

675. Seepage control measures are considered necessary where observed or estimated values of h_o may be expected to equal or exceed h_c (approximately 0.75 z_t) at design flood stages. If seepage control measures are considered necessary, they should be designed in accordance with the following criteria:

<u>a</u>. For levees with a semipervious top stratum landward of levee:

(1) <u>Riverside blankets.</u> Where no control measures are present, riverside blankets should be designed so that i at the toe of the levee does not exceed 0.5 to 0.6. Where landside berms wider than 150 ft are present, but additional control measures are considered necessary, riverside blankets should be designed so that i at the toe of the berm does not exceed 0.6 to 0.7.

(2) <u>Relief wells.</u> Where no control measures are present, relief wells should be designed so that i_{max} midway between wells or landward from the well line does not exceed 0.5 to 0.6. Where landside berms wider than 100 ft are present, but additional control measures are considered necessary, relief wells should be designed so that $i_{max} = 0.6$ to 0.7.

(3) <u>Seepage berms.</u> Seepage berms should have a width and thickness such that i through the top stratum and berm at the landside toe of the levee will not exceed 0.5, and i at the berm toe will not exceed 0.75 to 0.80. However, seepage berms need not have a width exceeding 300 to 400 ft depending on soil conditions and height of levee.

<u>b</u>. For levees with no natural top stratum landward of levees:

(1) <u>Riverside blankets.</u> If creep ratio is less than values in table 2 and Q_s at project flood stage would be excessive (say greater than about 200 gpm per 100 ft of levee), riverside blankets should be

designed to reduce Q_s to an acceptable amount.

(2) Relief wells. If creep ratio is too low and natural seepage Q_s is greater than about 200 gpm per 100 ft of levee, relief wells should be designed to intercept enough seepage so that the uncontrolled seepage emerging landward of the levee will not be more than about 150 to 200 gpm.

(3) Seepage berms. If creep ratio is less than values in table 2, length of berm should be such as to increase the creep ratio to an acceptable value, and i through the berm at toe of levee to a value equal to or less than 0.5.

676. The above-listed methods of underseepage control and their design for levees in the Mississippi River Valley are discussed in the following sections. The design of drainage blankets and trenches, cutoffs, and sublevees is also discussed as they may have some application in special cases along levees in alluvial valleys.

Riverside Blankets

Use of blankets

677. An impervious riverside blanket can be used to reduce the intensity of seepage and pressures landward of a levee where the pervious substratum is, or is nearly, exposed riverward of the levee. Such blankets are particularly adapted to situations where no top stratum exists riverward of the levee or where most of the natural top blanket has been removed in borrow operations. The primary purpose of a riverside blanket is to increase the distance from the levee to the point of seepage entry, thereby reducing both seepage and landward pressures. Riverside blankets can be placed by hauling in and compacting relatively impervious soils, or by constructing abatis dikes or encouraging willow growth to promote silting of borrow pits.

Design of blankets

678. Correct design of a riverside blanket requires determination of the extent, type, thickness, and permeability of the existing blanket. The first three of these items can be determined from surveys and borings; the permeability can then be estimated from values of k_{bR} in table 37.

679. Where the blanket is to be developed by means of abatis dikes,

the new fill will probably consist of silts or silty sands, depending upon velocity and flow conditions along the levee during high water. On the basis of data in table 37, the permeability of such a blanket would probably be about 1 to 2 x 10^{-4} cm per sec.

680. Where haul-in construction is contemplated, reasonable estimates of the permeability of blanket materials can be obtained from laboratory tests on compacted samples. The samples should be compacted to a density no greater than would be attained in the field.

681. Riverside blankets for control of underseepage should be designed so that the rate of seepage and head at the toe of the levee are acceptable at project flood stage. Formulas for the design of riverside blankets for various conditions are presented in the following paragraphs. The formulas for blankets of uniform thickness are based on blanket formulas given in Part III; formulas for triangular-shaped blankets are based on formulas given in reference 3. Values of x_3 in the following equations can be determined from piezometers or computed from the formulas given in fig. 23. Values of h_a should be based on observed values of h_c as obtained from seepage observations and piezometer data or computed from gradients recommended in paragraph 675. Required values of the effective length of riverside blanket x_r, thickness z_B, permeability k_B, and length L_B can be computed from the formulas and graphs given in the following paragraphs.

682. Case I. No natural riverside top stratum (fig. 48).

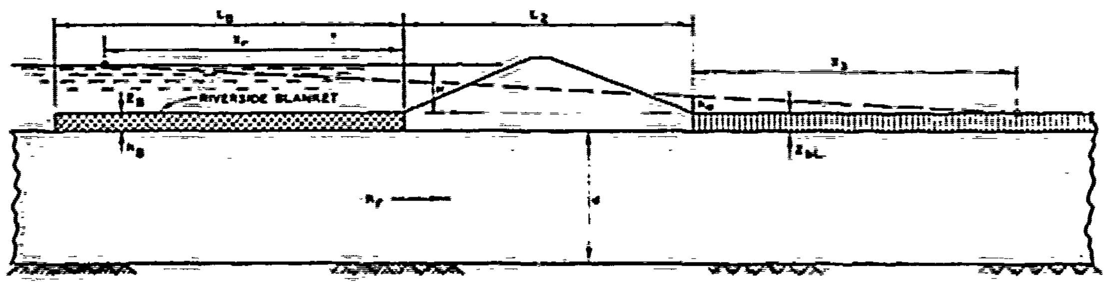


Fig. 48. Nomenclature for designing riverside blankets. No natural riverside top stratum

a. Blanket of uniform thickness:

$$x_r = x_3\left(\frac{H}{h_a} - 1\right) - L_2 \tag{35}$$

and

$$x_r = \frac{\tanh\left[L_B\sqrt{\frac{k_B}{k_f\ d\ z_B}}\right]}{\sqrt{\frac{k_B}{k_f\ d\ z_B}}} = \frac{\tanh c_B L_B}{c_B} . \qquad (36)$$

Various combinations of $\frac{z_B}{k_B}$ and L_B can be determined from fig. 49 for x_r. The best combination of z_B and L_B is that along the x_r curves near the dashed line. After selecting L_B

$$\frac{z_B}{k_B} = \frac{1}{k_f\ d\ c_B^2} . \qquad (37)$$

An example of the design of a riverside blanket based on Case 6 in fig. 23 for no riverside top stratum is given below:

for

H	$= 25$ ft	k_f	$= 1200 \times 10^{-4}$ cm per sec
L_2	$= 300$ ft (incl a 100-ft berm)	k_{bL}	$= 4 \times 10^{-4}$ cm per sec
L_3	$= 800$ ft	h_a	$= i_a\ z_{bL} = 0.7 \times 8 = 5.6$ ft
d	$= 80$ ft		
z_{bL}	$= 8$ ft (silt)	$\frac{k_f}{k_{bL}}$	$= 300$

From fig. 20, x'_3 (for $d = 100$ ft) $= 490$ ft for $L_3 = \infty$

For $d = 80$ ft, $x''_3 = 0.1\sqrt{d}\,x'_3 = 0.1\sqrt{80} \times 490 = 440$ ft ($L_3 = \infty$)

For $L_3 = 800$ ft,

$$\frac{x''_3 \text{ (for } L_3 = \infty)}{L_3} = \frac{440}{800} = 0.55 .$$

From fig. 21, factor (F) for computing x_3 for finite $L_3 = 1.06$

or

$$x_3 = (F)\ x''_3 = 1.06 \times 440 = 470 \text{ ft}$$

$$x_r = 470\left(\frac{25}{5.6} - 1\right) - 300 = 1330 \text{ ft} .$$

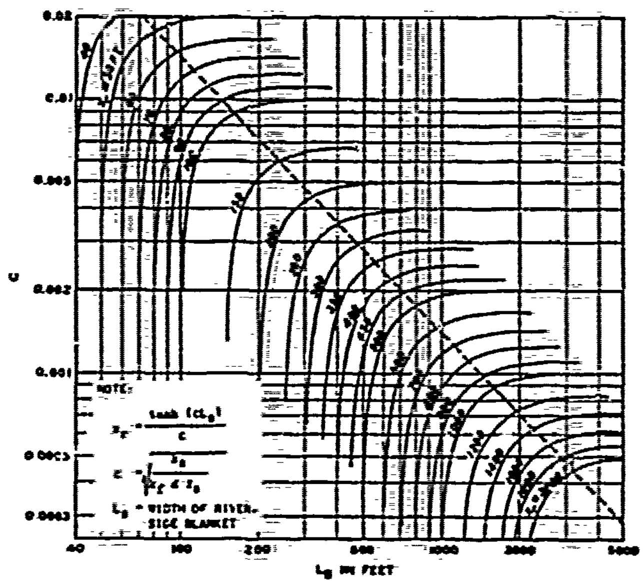


Fig. 49. Values of L_B and c for x_r. Finite length of riverside blanket on pervious substratum

From fig. 49, select $L_B = 1800$ ft
then

$$c_B = 0.0006$$

or

$$\frac{z_B}{k_B} = \frac{1}{1200 \times 10^{-4} \times 80 \times 0.0006^2} = 290{,}000 \ .$$

For $k_B = 0.1 \times 10^{-4}$ cm per sec (compacted silt), $z_B = 2.9$ ft.

683. As 3 ft is considered the minimum thickness permissible for a riverside blanket, the final design for this example would be $z_B = 3$ ft, $L_B = 1800$ ft, and $k_B = 0.1 \times 10^{-4}$ cm per sec. This blanket would have a volume of 20,000 cu yd per 100-ft levee station.

For

$L_B = 1400$ ft and $k_B = 0.01 \times 10^{-4}$ cm per sec (compacted lean clay),

$c_B = 0.0003$,

and

$$\frac{z_B}{k_B} = 1{,}150{,}000 \quad \therefore \quad z_B = 1.15 \text{ ft} .$$

Thus, a compacted blanket of 3-ft-thick (minimum) lean clay 1400 ft wide would be adequate for this assumed condition; it would have a volume of 15,500 cu yd per 100-ft levee station.

b. Blanket of triangular section:[3]

$$\frac{z_B}{k_B} = \frac{L_B\, x_r}{\left(\frac{L_B}{x_r} - 1\right) k_f\, d} \tag{38}$$

when

z_B = thickness of blanket at levee

L_B = length of triangular blanket.

An example of the design of a triangularly shaped riverside blanket based on the same pervious substratum and landward conditions as given in paragraph 682 is given below:

Assume: $L_B = 1800$ ft

$$\frac{z_B}{k_B} = \frac{1800 \times 1330}{\left(\frac{1800}{1330} - 1\right) 1200 \times 10^{-4} \times 80} = 715{,}000$$

(1) For $L_B = 1800$ ft and $k_B = 0.1 \times 10^{-4}$ cm per sec, $z_B = 7.2$ ft

(2) For $L_B = 1400$ ft and $k_B = 0.01 \times 10^{-4}$ cm per sec, $z_B = 3.9$ ft.

The volume of blanket (1) would be 23,800 cu yd per 100-ft levee station; the volume of blanket (2) would be 13,000 cu yd per 100-ft levee station as 5 ft is considered the minimum z_B for a triangular blanket.

684. Case II. Existing natural uniform top stratum and blanket from levee to river (fig. 50).

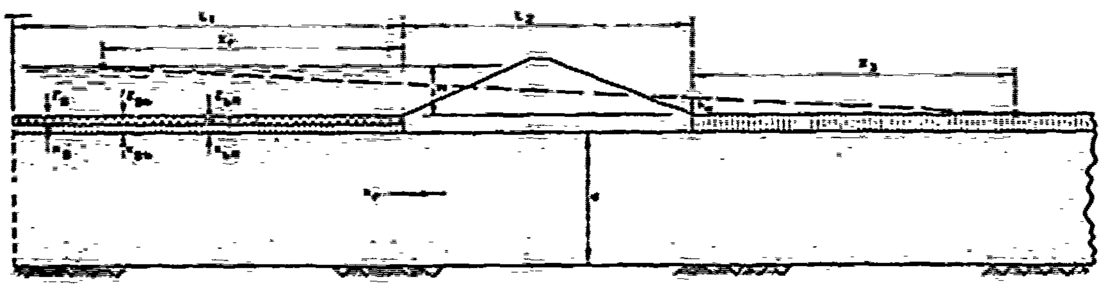


Fig. 50. Nomenclature for designing riverside blankets. Natural uniform top stratum from levee to river

$$x_r = x_3\left(\frac{H}{h_a} - 1\right) - L_2 \tag{35}$$

and

$$x_r = \frac{\tanh\left[L_1 \sqrt{\frac{k_{Bb}}{k_f\ d\ z_{Bb}}}\right]}{\sqrt{\frac{k_{Bb}}{k_f\ d\ z_{Bb}}}} = \frac{\tanh c_{Bb} L_1}{c_{Bb}} \tag{36a}$$

where

$$L_B = L_1$$

$$z_{Bb} = z_b + z_B$$

$$k_{Bb} = \frac{z_b + z_B}{\frac{z_b}{k_{bR}} + \frac{z_B}{k_B}} \ .$$

From fig. 49, obtain z_{Bb} for L_1, then

$$\frac{z_{Bb}}{k_{Bb}} = \frac{1}{k_f\ d\ c_{Bb}^{\ 2}} \ . \tag{37a}$$

685. <u>Case III. Natural top stratum riverward of borrow pit assumed infinite (L_1 > 2000 ft) and to have same characteristics as top stratum and uniform blanket in borrow pit (fig. 51).</u>

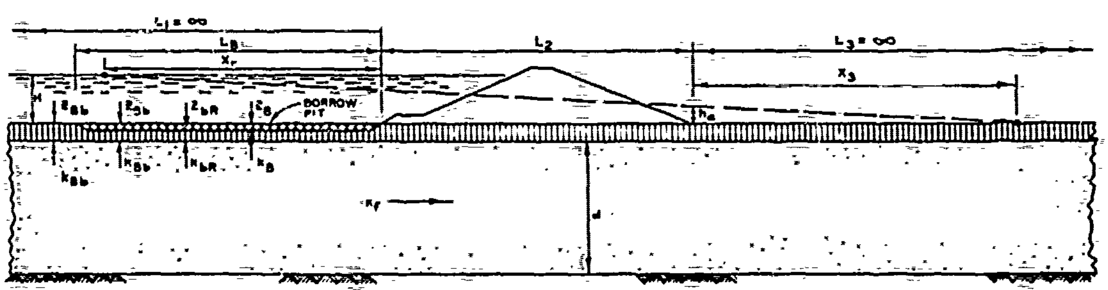


Fig. 51. Nomenclature for designing riverside blankets in borrow pits. Natural top stratum riverward of levee infinite and same as top stratum and blanket in borrow pit

$$x_r = x_3 \left(\frac{H}{h_a} - 1\right) - L_2 \tag{35}$$

Assume

$$z_{Bb} = z_{bR} + z_B$$

Assume

$$k_{Bb} = \frac{z_{bR} + z_B}{\frac{z_{bR}}{k_{bR}} + \frac{z_B}{k_B}} \tag{3a}$$

$$\frac{z_B}{k_B} = \frac{x_r^2}{k_f d} - \frac{z_{bR}}{k_{bR}} \ . \tag{39}$$

Example: Same pervious substratum and landward conditions as example on page 279.

Example	Natural Top Stratum and Borrow Pit Blanket	z_{bR}	k_{bR} and k_B
(a)	Sandy silt	4 ft	1.5×10^{-4} cm/sec
(b)	Stratified silt	4 ft	0.5×10^{-4} cm/sec
(c)	Clay silt	4 ft	0.1×10^{-4} cm/sec

$$x_r = 1330 \text{ ft}$$

$$\frac{z_B}{k_B} = \frac{1330^2}{1200 \times 10^{-4} \times 80} - \frac{4}{k_{bR} \times 10^{-4}} = 157{,}000 .$$

(a) For k_{bR} and $k_B = 1.5 \times 10^{-4}$ cm per sec (natural river deposit of silt) $z_B = 23.5$ ft

(b) For k_{bR} and $k_B = 0.5 \times 10^{-4}$ cm per sec (compacted blanket of silt) $z_B = 7.8$ ft

(c) For k_{bR} and $k_B = 0.1 \times 10^{-4}$ cm per sec (compacted blanket of clay silt) $z_B = 1.6$ ft.

Thus, for the pervious substratum and landward conditions assumed in the above examples, filling a riverside borrow pit with a soil having a $k_B = 1.5 \times 10^{-4}$ cm per sec would not be a practical seepage control measure where $k_{bR} = 1.5 \times 10^{-4}$ cm per sec. Whether or not the blanket thicknesses computed in examples (b) and (c) would be practical would depend upon the thickness and permeability of the natural top stratum riverward of the borrow pit and the width of the borrow pit.

686. Case IV. Natural top stratum riverward of borrow pit assumed infinite ($L_1 > 2000$ ft) and impervious ($k < 0.05 \times 10^{-4}$ cm per sec) with a uniform blanket in borrow pit (fig. 52).

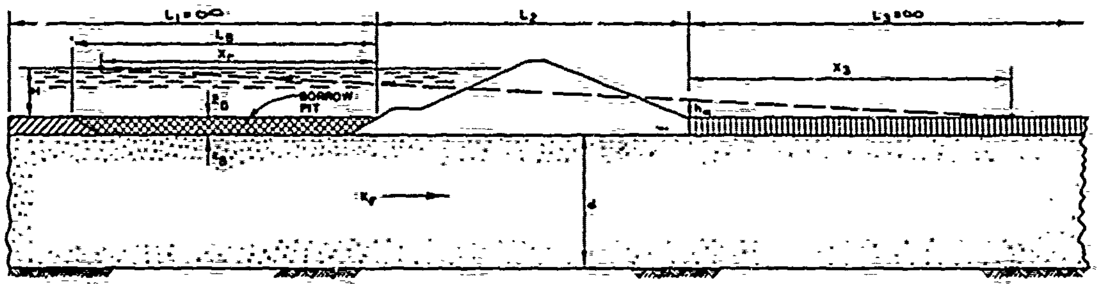


Fig. 52. Nomenclature for designing riverside blankets. Top stratum riverward of borrow pit infinite and impervious

$$x_r = x_3\left(\frac{H}{h_a} - 1\right) - L_2 \tag{35}$$

and

$$x_r = \frac{1}{\sqrt{\frac{k_B}{k_f\ d\ z_B}}\ \tanh\left[L_B\sqrt{\frac{k_B}{k_f\ d\ z_B}}\right]} = \frac{1}{c_B\ \tanh\ c_B\ L_B} \ . \tag{40}$$

687. The value of c_B or $\sqrt{\frac{k_B}{k_f\ d\ z_B}}$ can be obtained from fig. 53 for any given width of borrow pit. From c_B

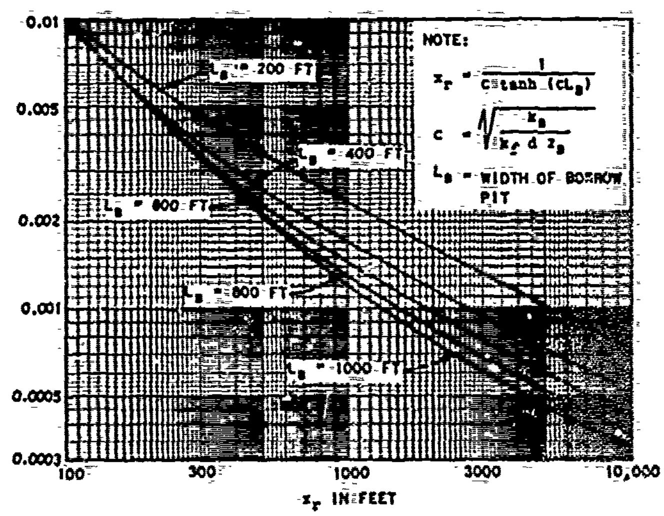


Fig. 53. Values of L_B and c for x_r. Top stratum riverward of borrow pit impervious and infinite in extent

$$\frac{z_B}{k_B} = \frac{1}{k_f\ d\ c_B^{\ 2}} \tag{37}$$

<u>Example:</u> Same pervious substratum and landward condition as example on page 279.

$$L_B = 500 \text{ ft}$$

$$x_r = 1330 \text{ ft}$$

From fig. 53,

$$c_B = 0.00133$$

or

$$\frac{z_B}{k_B} = \frac{1}{1200 \times 10^{-4} \times 80 \times (0.00133)^2}$$

(a) For $k_B = 1.5 \times 10^{-4}$ cm per sec (natural river deposit of silt)
$z_B = 8.8$ ft

(b) For $k_B = 0.5 \times 10^{-4}$ cm per sec (compacted blanket of silt)
$z_B = 2.9$ ft

(c) For $k_B = 0.1 \times 10^{-4}$ cm per sec (compacted blanket of clay silt)
$z_B = 0.6$ ft.

688. If in the above example a uniform natural blanket had been present in the borrow pit and an additional blanket were considered necessary, the procedure used above would yield the total blanket thickness z_{Bb} and average permeability k_{Bb}. The required thickness z_B and permeability k_B of the new blanket can then be obtained from the thickness and permeability of the existing blanket and equation 3a.

689. From the previous formulas and numerical examples it can be seen that, if the permeability of the blanket is less than about 0.1 x 10^{-4} cm per sec, a minimum thickness of blanket (3 to 5 ft) will usually be adequate. To obtain such a permeability, the blanket would necessarily be of compacted clay or clay silt. Natural blankets resulting from siltation by means of abatis dikes and willow growth will result in an increase in x_1 and s, but may not suffice to reduce h_o to h_a because such a blanket may be fairly pervious (k_B = 1 to 2 x 10^{-4} cm

per sec). Blankets developed by means of abatis dikes may be adequate where h_o without the blanket does not greatly exceed h_a .

690. According to Bennett[3] a triangular-shaped blanket will tend to be about 25 per cent more efficient for the same length and volume of material as a uniform blanket of constant thickness. Triangular blankets are desirable where long blankets are contemplated but probably would not be used in narrow riverside borrow pits. Procedures for designing triangular blankets are given in reference 3.

Relief Wells

Use of relief wells

691. Relief wells of proper spacing and penetration can be used to reduce excess hydrostatic pressure landward of levees underlain by a pervious foundation for a wide range of seepage entrances, foundation conditions, and landward top strata. The primary purpose of relief wells is to reduce artesian pressures above the ground surface which otherwise would cause formation of sand boils and possibly subsurface piping. Properly designed wells also reduce substratum pressures for a sufficient distance landward of the levee to preclude the possibility of dangerous seepage landward of the line of wells. Relief wells also intercept and provide controlled outlets for seepage which otherwise would emerge uncontrolled landward of the levee.

692. Relief wells should be designed to penetrate into the principal pervious strata to obtain efficient pressure relief, especially where the foundation is stratified. Wells should be spaced sufficiently close together to intercept seepage and reduce to safe values hydrostatic pressure which otherwise would act beyond the wells. Wells must offer little resistance to water flowing through the screen and out of the well; they must prevent infiltration of sand into the well after initial development; and they must be able to resist the deteriorative action of water, soil, and bacteria.

693. Disadvantages of relief wells are that they require periodic inspection and maintenance (see Part VIII), they must be protected from

backflooding, and they increase the total quantity of seepage about 20 to 40% depending on conditions. However, these disadvantages can partially be overcome by providing a suitable well guard, check valve and rubber gasket, and standpipe to prevent the wells from flowing at low flood heights.

694. The principles of controlling seepage by relief wells are illustrated by figs. 54-58. These data were obtained from sand models built to simulate typical conditions along Mississippi River levees. (Additional data from these model studies are presented in Appendix A; results of full-scale test on a relief well system at Trotters 54 are summarized in Appendix D.) The effects of well spacing and penetration on seepage, well flows, and substratum pressures in a homogeneous sand foundation are illustrated for various landside top strata in figs. 54-56. The effects of stratification and borrow pit conditions on the operation of well systems are illustrated in figs. 57 and 58. It is apparent from figs. 57 and 58 that wells should penetrate the more pervious strata of the substratum in order to efficiently relieve substratum pressures. From fig. 56 it is seen that relief wells increase the total quantity of seepage somewhat, although they materially reduce the natural seepage through the landside top stratum (e.g., wells on 150-ft spacing increased the total seepage 25% but decreased the seepage emerging through the top stratum by 75%). As only about 20 to 40% of the levees along the Middle and Lower Mississippi River are possibly critical with respect to dangerous substratum pressures, the total seepage flow along the levee system would not be increased by more than about 10 to 20%. Also, the flow resulting from surface runoff and general seepage during flood that must be accommodated by any existing surface drainage system landward of the levee is appreciable without relief wells. The increase in this flow that would be caused by the installation of relief wells along certain critical reaches of the levee would be relatively small during high water.

695. Pertinent factors to be considered in the design of well systems are well radius, well spacing, depth and permeability of the foundation, stratification of the foundation, distance to the effective source of seepage, characteristics of the landside top stratum, net head

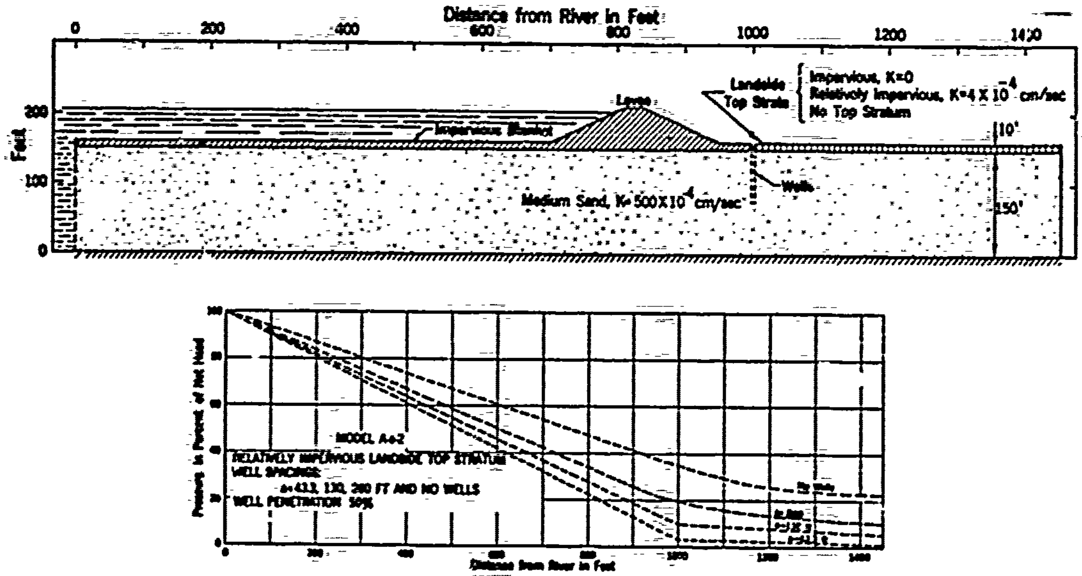


Fig. 54. Hydraulic grade line beneath top stratum with and without relief wells. Homogeneous sand foundation and relatively impervious landside top stratum (Model A)

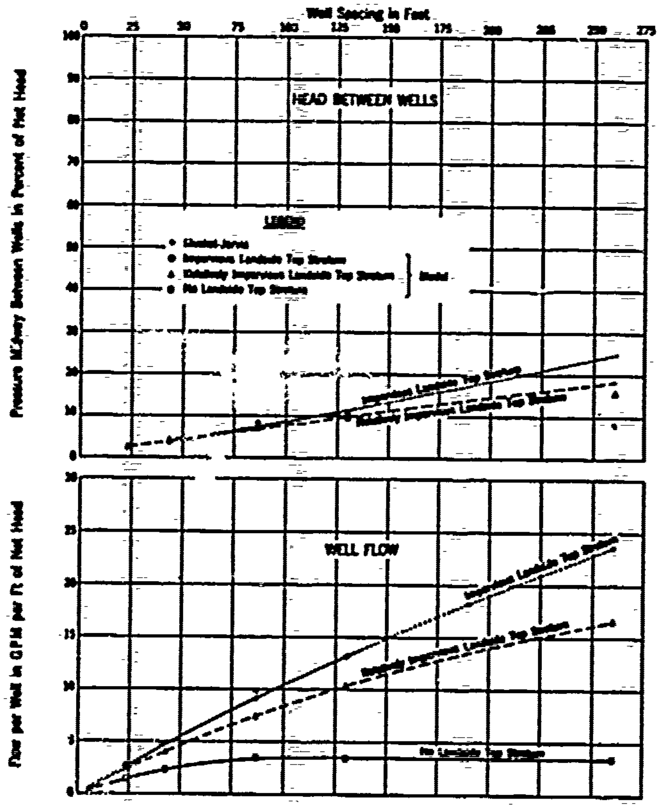


Fig. 55. Well flows and landside substratum pressures. Homogeneous sand foundation, various landward top strata (Model A). Well penetration, 50%

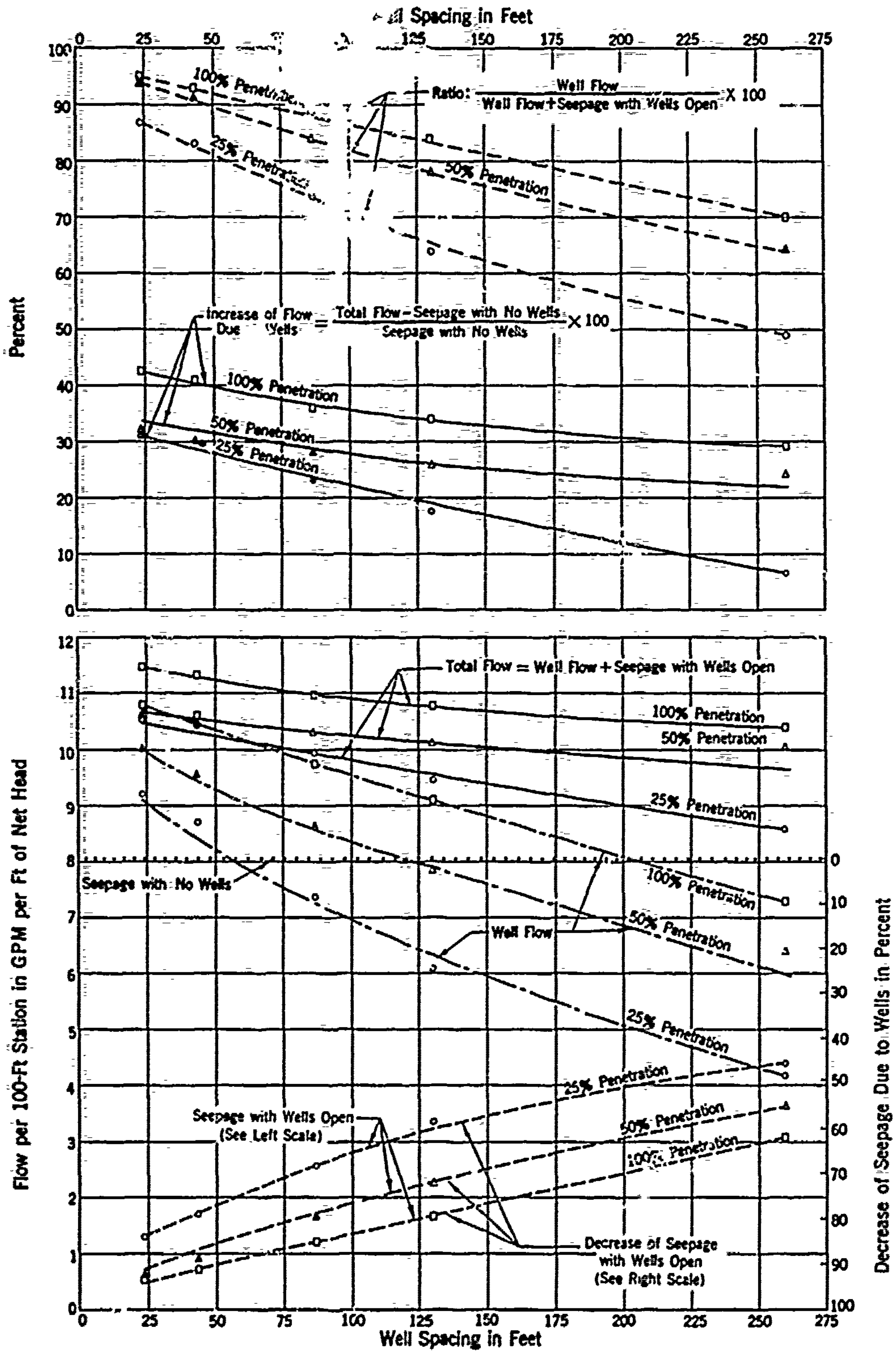


Fig. 56. Well flow and seepage. Homogeneous sand foundation and relatively impervious landside top stratum (Model A)

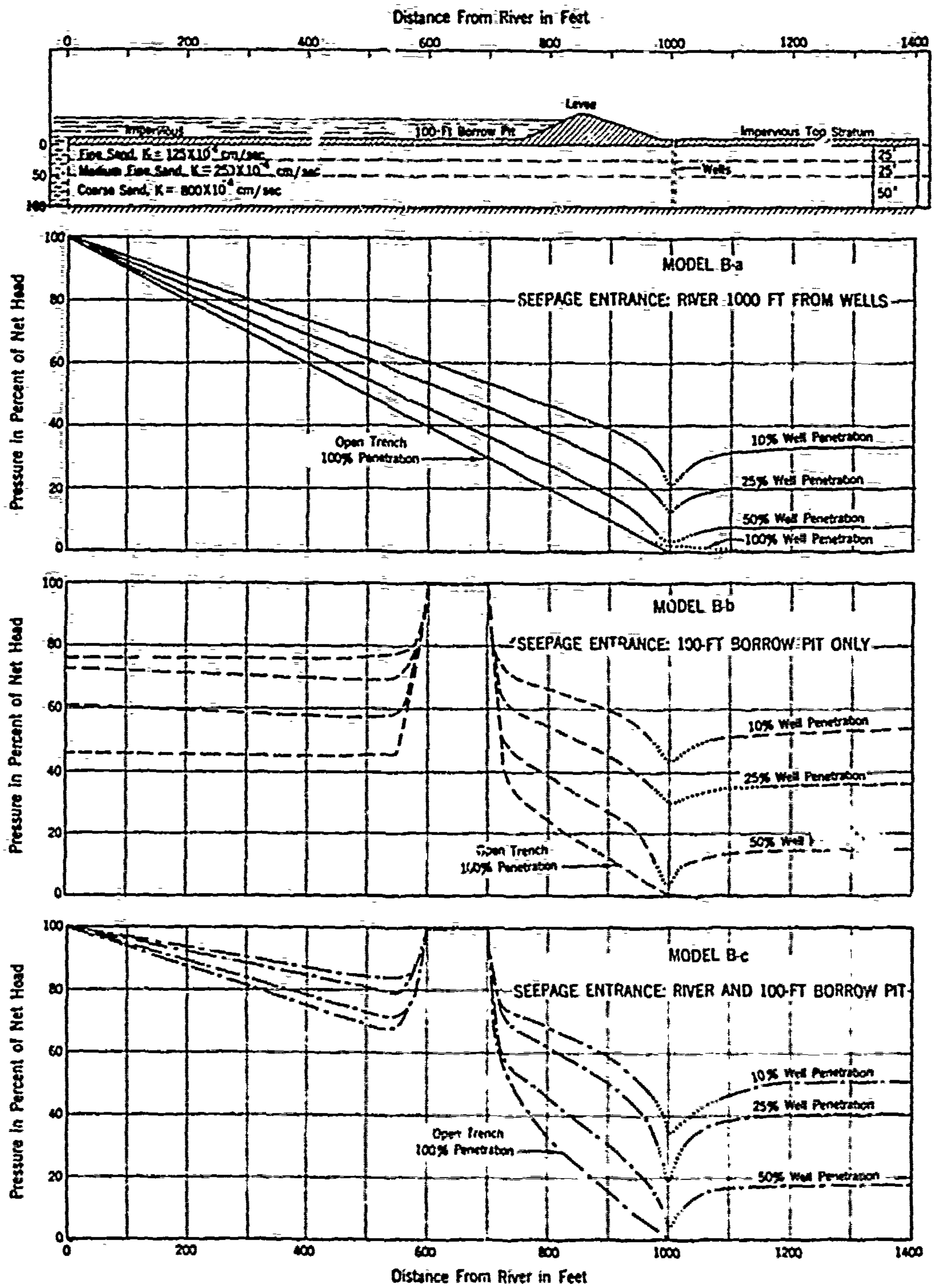


Fig. 57. Hydraulic grade line beneath top stratum with relief wells and various seepage entrances. Stratified sand foundation and impervious landside top stratum (Model B)

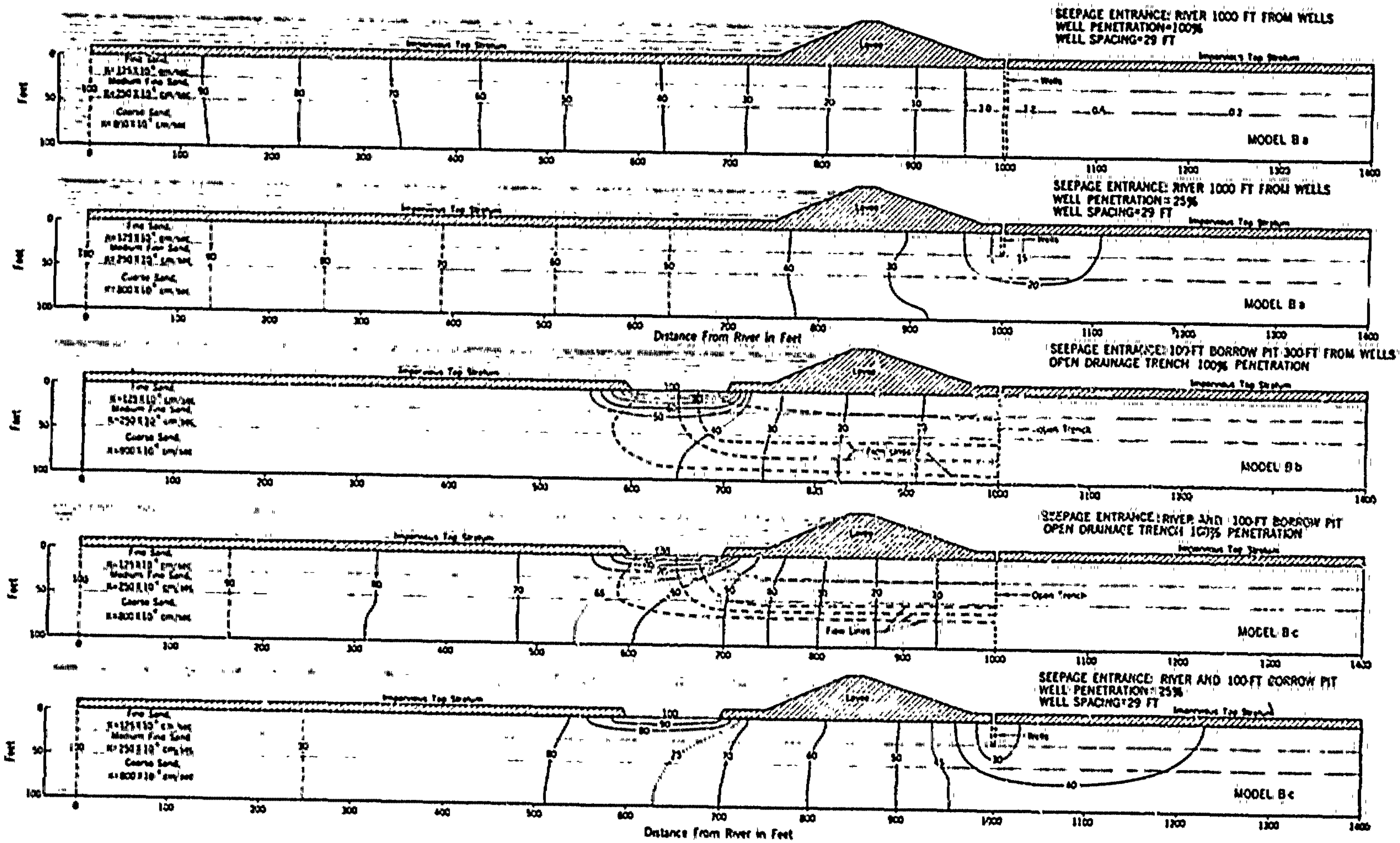


Fig. 58. Equipotential and flow lines with relief wells and various seepage entrances. Stratified sand foundation and impervious landside top stratum (Model B)

on the levee, and degree of pressure relief or seepage interception desired.

Design of the well

696. The design of the well itself consists of the selection of type and length of riser pipe and screen, design of the gravel filter, and design of relief well appurtenances. Treated wood-stave riser and screen is economical and noncorrosive, and is recommended for relief wells. The uppermost 10 to 15 ft of the riser pipe should be surrounded by concrete backfill to insure against decay resulting from fluctuations in ground-water level. To prevent filter gravel from entering the well and to minimize screen entrance head losses, the slots in the well screen must have adequate area and yet be of such size as to prevent movement of filter through the screen after development of the well (see criteria below).

$$\frac{(\text{Min})D_{85}\text{ Filter}}{\text{Slot width}} \geqq 1.2 \quad \text{or} \quad \frac{(\text{Min})D_{85}\text{ Filter}}{\text{Hole diam}} \geqq 1.0$$

The gradation of the filter must also comply with the following criteria:

$$\frac{(\text{Max})D_{15}\text{ Filter}}{(\text{Min})D_{85}\text{ Sand}} \leqq 5.0 \quad \text{and} \quad \frac{(\text{Min})D_{15}\text{ Filter}}{(\text{Max})D_{15}\text{ Sand}} \geqq 4 .$$

Wooden screens for relief wells are commercially available with 3/16- x 3-1/4-in. slots and with open area of the slots equal to 10% of the circumferential area of the screen.

697. Wells in the alluvial valley of the Mississippi River should have an inside diameter of 8 in. in order that their carrying capacity will be adequate without excessive head loss in the well. An 8-in. well with a 6-in. gravel filter has an effective radius of about 0.8 to 1.0 ft.

698. In a stratified foundation, the effective well penetration usually differs from that computed from the ratio of the length of well screen to total thickness of the aquifer. The effective screen penetration W of a well screen length $\bar{W}$ in a stratified foundation can be determined in the following manner. Each stratum of the pervious substratum with thickness d_n and horizontal and vertical permeability

coefficients k_{H-n} and k_{V-n} can be transformed into an isotropic layer of thickness $\bar{d}_n$ and permeability $\bar{k}_n$ by means of the following equations:

$$\bar{d}_n = d_n \sqrt{\frac{k_{H-n}}{k_{V-n}}}$$

$$\bar{k}_n = \sqrt{k_{H-n}\, k_{V-n}} .$$

The thickness of the transformed, homogeneous, isotropic foundation $\bar{D}$ is

$$\bar{D} = \sqrt{\Sigma\,(d_n\, k_{H-n})\ \Sigma\,(d_n/k_{V-n})}$$

and the effective permeability of the transformed foundation $\bar{K}$ is

$$\bar{K} = \sqrt{\frac{\Sigma\,(d_n\, k_{H-n})}{\Sigma\,(d_n/k_{V-n})}} .$$

The effective well screen penetration W into the transformed foundation is

$$W = \frac{\sum_{0}^{\bar{W}} d_n\, k_{H-n}}{\bar{K}} .$$

The per cent penetration of the well screen in the transformed foundation is

$$\left(\frac{W}{\bar{D}}\right)\% = \frac{100 \sum_{0}^{\bar{W}} d_n\, k_{H-n}}{\bar{K}\,\bar{D}} = \frac{100 \sum_{0}^{\bar{W}} d_n\, k_{H-n}}{\sum_{0}^{D} d_n\, k_{H-n}} .$$

699. Along the middle portion of the Mississippi River where the substratum tends to become more pervious with depth and where the effective thickness averages about 100 ft, it has been found from pumping tests that to achieve an effective penetration of 50%, the wells should penetrate about 60% of the principal seepage carrying aquifer on a length basis.[54] This degree of penetration usually results in wells about 75 to

110 ft deep. The principal seepage carrying aquifer is considered to be the strata of sands below the upper top strata of clays, silts, and fine sands and above the valley floor. A depth of about 125 ft represents about the economical limit for well installation; and, therefore, a 50% penetrating system is about the practical maximum that can be achieved along Mississippi River levees. In general, it is believed that relief wells along Mississippi River levees should be designed on the basis of an effective penetration of about 50% of the main sand aquifer.

700. To prevent wells from becoming backflooded with muddy surface water (which greatly impairs their efficiency) when they are not flowing, a check valve and rubber gasket should be installed on each well. A simple, inexpensive aluminum check valve and rubber gasket have been found to effectively protect wells from backflooding both in the St. Louis District and in simulated field tests at the Waterways Experiment Station. As a safeguard against animals, vandalism, or accidental damage, the tops of the relief wells should be provided with a metal well guard to protect the check valve and standpipe and to prevent the entrance of debris. These devices will greatly reduce required maintenance. To prevent wells from discharging when there is relatively little head on the levee and no pressure relief is necessary, plastic standpipes can be used to raise the discharge elevation of the wells. The maximum height of the standpipe should not exceed 0.25 h_a; they should be removed when the hydrostatic head in the foundation causes them to overflow. These appurtenances are illustrated in Part VII.

Design formulas

701. Formulas for designing relief wells have been developed from theoretical and model studies,[1,23] but until recently these formulas were limited to fully penetrating wells with either an impervious or leaking top stratum, or partially penetrating wells with an impervious top stratum. For conditions encountered along the Lower Mississippi River, formulas are required for designing partially penetrating wells with a leaking top stratum. A set of design curves for this case was devised in 1952 by the Waterways Experiment Station by combining formulas and graphs given in references 1 and 23. These curves were used to design the relief well

systems recently installed in the St. Louis District. Since then, electrical analogy model studies have been conducted under Civil Works Investigation No. 510 to obtain design curves for partially penetrating wells with a leaking top stratum; design graphs and procedures resulting from this investigation are given in OCE Civil Works Bulletin 55-11, Relief Well Design.[26] Design curves and procedures contained herein are based on data given in this bulletin.

702. Landside top stratum. The design of a well system consists essentially of determining the spacing and penetration of wells that will reduce the substratum pressure h_o at the toe of the levee to an allowable head h_a. The well spacing is first determined assuming an infinite line of wells, and then the spacing is reduced, where necessary, to allow for the reduced efficiency of a finite line of wells as compared to an infinite line. For given values of h_a, H, s, k_f, and x_3, there are any number of combinations of well spacing and penetration that will suffice; the final selected spacing and penetration should be based, to an extent, on the most economical design.[26]

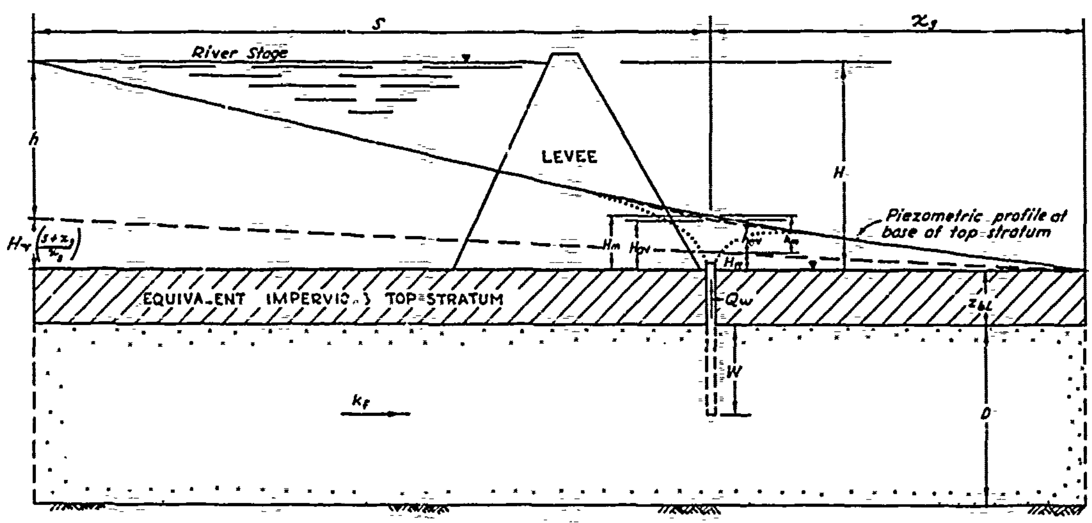
703. The nomenclature and equations for design of relief wells given in fig. 59 are for an infinite line of wells penetrating into a homogeneous, isotropic (either natural or transformed) pervious substratum overlain by a leaking top stratum. To determine the required spacing of an infinite line of wells of given W and W/D, it is necessary to utilize the following procedure of successive trials, and the nomograph for well design in fig. 60. The required well spacing is affected by hydraulic head losses in the well; these losses which consist of screen entrance loss, friction loss, and velocity head loss can be estimated for an 8-in. ID wood-stave well from fig. 61. The procedure for computing the well spacing is outlined below.

a. Compute h_a from $h_a = i_o z_t$.

b. Assume that $H_{av} = h_a$ and compute ΔM from equation 46.

c. Assume a well spacing a and compute Q_w from equation 47.

d. Estimate H_w for the above Q_w and W/D by means of fig. 61.



NOTATIONS

h = EFFECTIVE NET HEAD ON THE WELL SYSTEM
H = TOTAL NET HEAD ON THE LEVEE
h_a = ALLOWABLE HEAD BENEATH TOP STRATUM
H_w = TOTAL HEAD LOSS IN WELL INCLUDING ELEVATION HEAD LOSS
H_m = NET HEAD BENEATH TOP STRATUM MIDWAY BETWEEN WELLS
H_{av} = NET HEAD IN THE PLANE OF WELLS
h_{av} = NET AVERAGE HEAD IN THE PLANE OF WELLS ABOVE H_w
h_m = NET HEAD BENEATH TOP STRATUM MIDWAY BETWEEN WELLS ABOVE H_w
θ_{av} = AVERAGE UPLIFT FACTOR
θ_m = MIDPOINT UPLIFT FACTOR
ΔM = NET SEEPAGE GRADIENT TOWARD THE WELL LINE
a = WELL SPACING
r_w = EFFECTIVE WELL RADIUS
W = EFFECTIVE LENGTH OF WELL SCREEN
D = EFFECTIVE THICKNESS OF PERVIOUS SUBSTRATUM
Q_w = FLOW FROM A SINGLE WELL

FORMULAS

$$h = H - H_w\left(\frac{s + x_3}{x_3}\right) \quad (41)$$

$$h_{av} = \frac{h\,\theta_{av}}{\frac{s}{a} + \left(\frac{s + x_3}{x_3}\right)\theta_{av}} \quad (42)$$

$$H_{av} = H_w + h_{av} \quad (43)$$

$$h_m = h_{av}\frac{\theta_m}{\theta_{av}} = \frac{h\,\theta_m}{\frac{s}{a} + \left(\frac{s + x_3}{x_3}\right)\theta_{av}} \quad (44)$$

$$H_m = H_w + h_m \quad (45)$$

$$\Delta M = \frac{H - H_{av}}{s} - \frac{H_{av}}{x_3} \quad (46)$$

$$Q_w = \frac{h\,k_f\,D}{\frac{s}{a} + \left(\frac{s + x_3}{x_3}\right)\theta_{av}} = a\,\Delta M\,k_f\,D \quad (47)$$

$$h_{av} = a\,\Delta M\,\theta_{av} \quad (48)$$

$$h_m = a\,\Delta M\,\theta_m \quad (49)$$

Fig. 59. Nomenclature and formulas for design of relief wells

e. Compute h_{av} from equation 43.

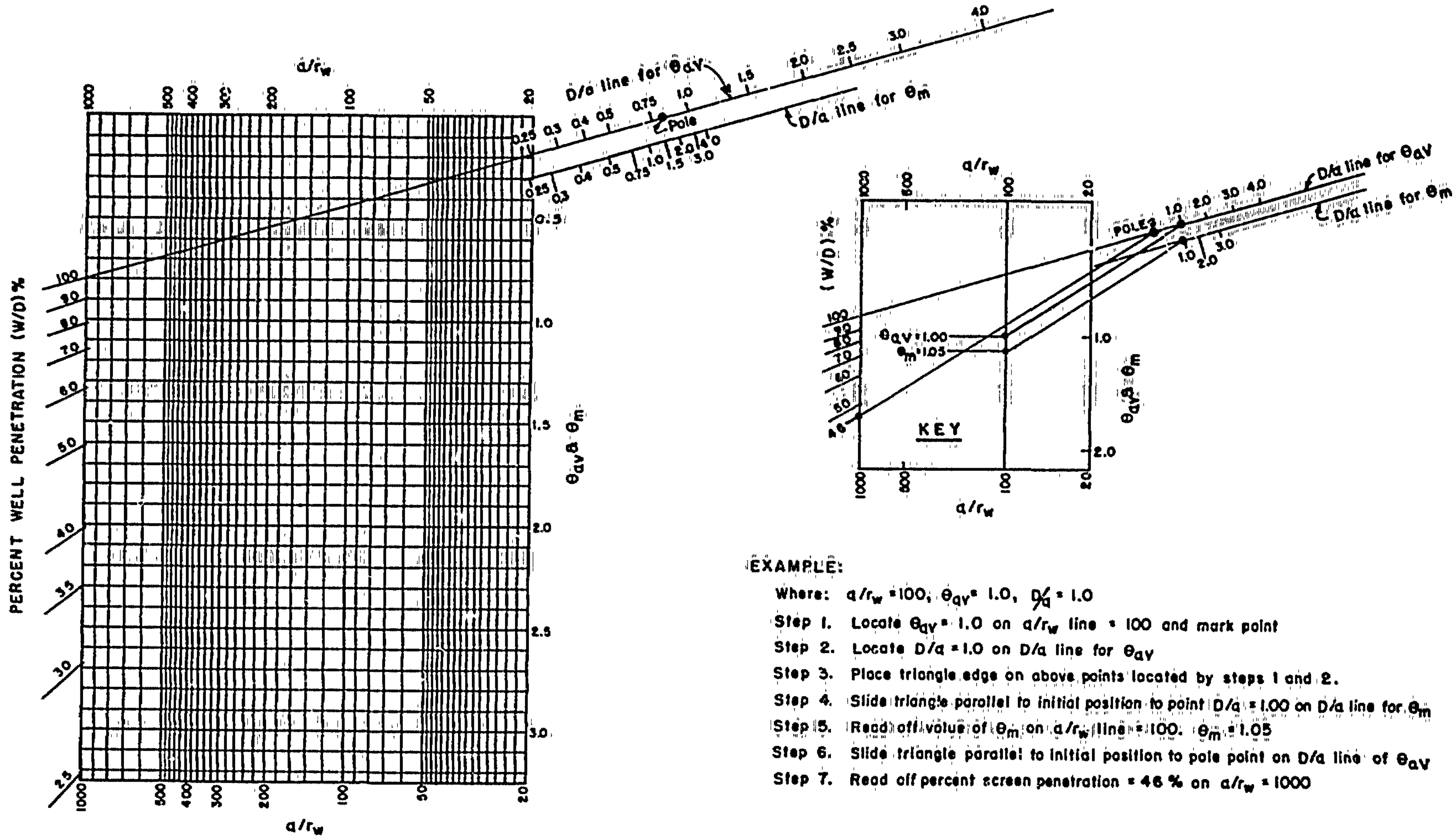
f. Substitute the above values of h_{av} and ΔM in equation 48 and solve for θ_{av} for various values of a.

g. Find θ_{av} from fig. 60 for the values of a used in step (f) and the corresponding a/r_w and D/a values.

h. The first trial well spacing is that of value a for which θ_{av} from step (f) = θ_{av} from step (g).

i. Find θ_m from fig. 60 for the first trial well spacing and the corresponding values of a/r_w and D/a.

j. If $\theta_{av} > \theta_m$, repeat procedure steps (c) to (i) inclusive using the first trial well spacing in lieu of the spacing originally assumed in step (c), and determine the second trial well spacing. The above procedure should



EXAMPLE:

Where: $a/r_w = 100$, $\theta_{av} = 1.0$, $D/a = 1.0$

Step 1. Locate $\theta_{av} = 1.0$ on a/r_w line = 100 and mark point

Step 2. Locate $D/a = 1.0$ on D/a line for θ_{av}

Step 3. Place triangle edge on above points located by steps 1 and 2.

Step 4. Slide triangle parallel to initial position to point $D/a = 1.00$ on D/a line for θ_m

Step 5. Read off value of θ_m on a/r_w line = 100. $\theta_m = 1.05$

Step 6. Slide triangle parallel to initial position to pole point on D/a line of θ_{av}

Step 7. Read off percent screen penetration = 46 % on $a/r_w = 1000$

Fig. 60. Nomographic chart for design of relief well systems

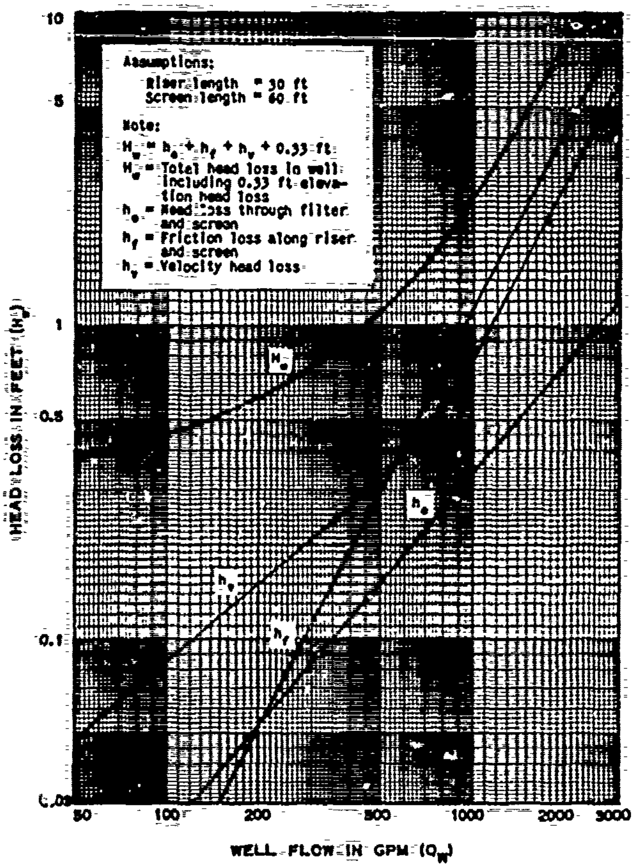


Fig. 61. Hydraulic head losses in 8-in. ID wood-stave well with 6-in. gravel filter

be repeated until relatively consistent values of a are obtained on two successive trials, although usually the second trial spacing is sufficiently accurate.

If in procedure step (j), $\theta_{av} < \theta_m$, then the following procedure should be used:

k. Assume that $H_m = h_a$ and compute Q_w from equation 47, using the value of $a\Delta M$ previously obtained in step (b) and the first trial well spacing previously found from step (h).

l. Estimate H_w from Q_w of step (k) and W/d , by means of fig. 61.

m. Compute h_m from equation 45 from H_w obtained in step (l).

n. Compute h_{av} from equation 44 using h_m from step (m) and θ_{av} and θ_m from steps (h) and (i), respectively.

o. From the above values of h_{av} and H_w, compute H_{av} from equation 43.

p. Compute ΔM from equation 46 using H_{av} from step (o).

q. Substitute the above values of h_m and ΔM in equation 49 and solve for Θ_m for various values of a.

r. Find Θ_m from fig. 60 for the values of a used in step (q) and the corresponding a/r_w and D/a values.

s. The second trial well spacing is that value of a for which Θ_m from step (q) = Θ_m from step (r).

t. Find Θ_{av} from fig. 60 for the second trial well spacing and the corresponding values of a/r_w and D/a.

u. Determine the third trial well spacing by repeating steps (k) to (t) inclusive, using the second trial well spacing in lieu of the spacing originally assumed in step (k), and in step (n) using the values of Θ_m and Θ_{av} from steps (s) and (t), respectively, instead of those from steps (h) and (i). This procedure should be repeated until relatively consistent values of a are obtained on two successive trials. Normally it will be found that the third trial spacing will be sufficiently accurate for design purposes.

704. In a short, finite line of wells, the heads midway between wells exceed those obtained for an infinite line of relief wells both at the center and near the ends of the well system. Numerous well systems may be fairly short (less than 1200 ft in length), and for these it will be necessary to reduce the well spacing computed for an infinite line of wells so that heads midway between wells will not be more than h_a. The ratio of the head midway between wells at the center of finite well systems to the head between wells in an infinite line of wells is shown in fig. 62 for various well spacings and exit lengths. The spacing of wells in a finite line should be the same as that required in an infinite line of wells to reduce the head midway between wells to h_a divided by ratio of $\frac{H_{m_N}}{H_{m_\infty}}$ from fig. 62. In any finite line of wells of constant penetration and spacing, the head midway between wells near the ends of the system exceeds that at the center of the system. Thus, at the ends of both short and long well systems, the wells should generally be made

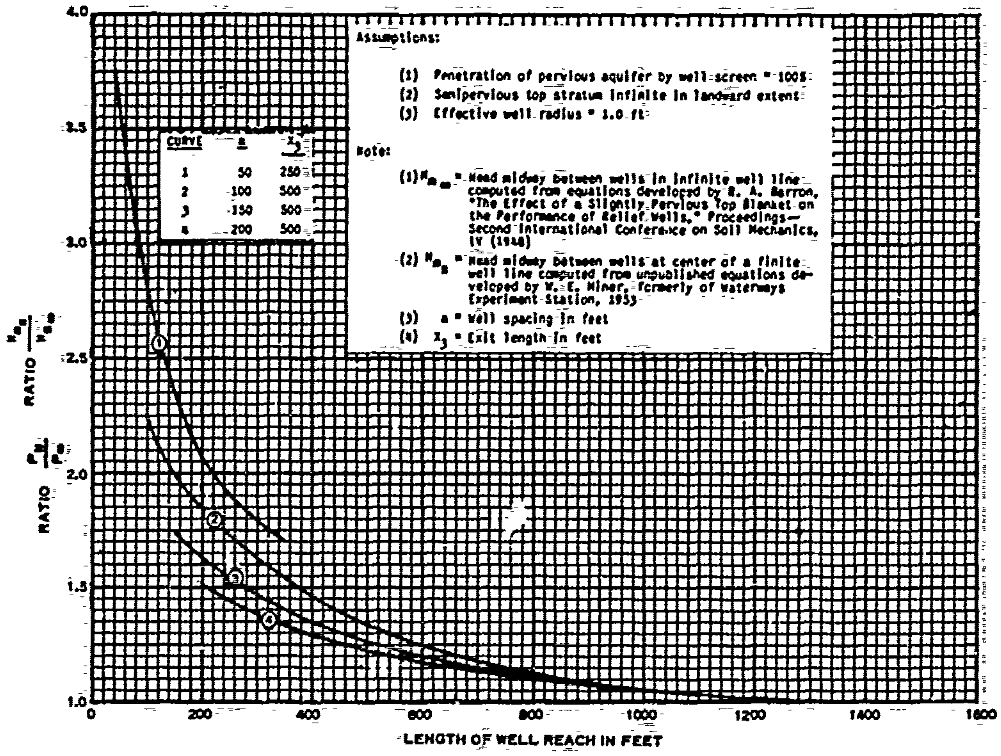


Fig. 62. Ratio of head midway between wells at center of a finite well system to head midway between wells in an infinite line of wells

deeper to provide additional penetration of the pervious substratum so as to obtain the same head reduction as in the central part of the well line.

705. After the well spacing for a given reach of levee has been determined, the location of each well should be checked in the office and in the field and adjusted where necessary so that the wells will be located at critical seepage spots and will fit natural topographic features.

706. A set of design curves generally applicable for designing relief well systems for levees along the Lower Mississippi River has been developed for average foundation conditions and for distances to the effective source of seepage of 500, 1000, 1500, and 2000 ft (see figs. 63-66). The curves are for wells with an effective penetration of 50%, $r_w = 1$ ft and $D = 100$ ft . The heads midway between wells in per cent are based on θ_{av} or θ_m , whichever is greater, and are valid for H = about 20 to 35 ft. The well flows are based on $k_f = 1250 \times 10^{-4}$ cm/sec, or about the average for sites above L'Argent, La., as the most critical

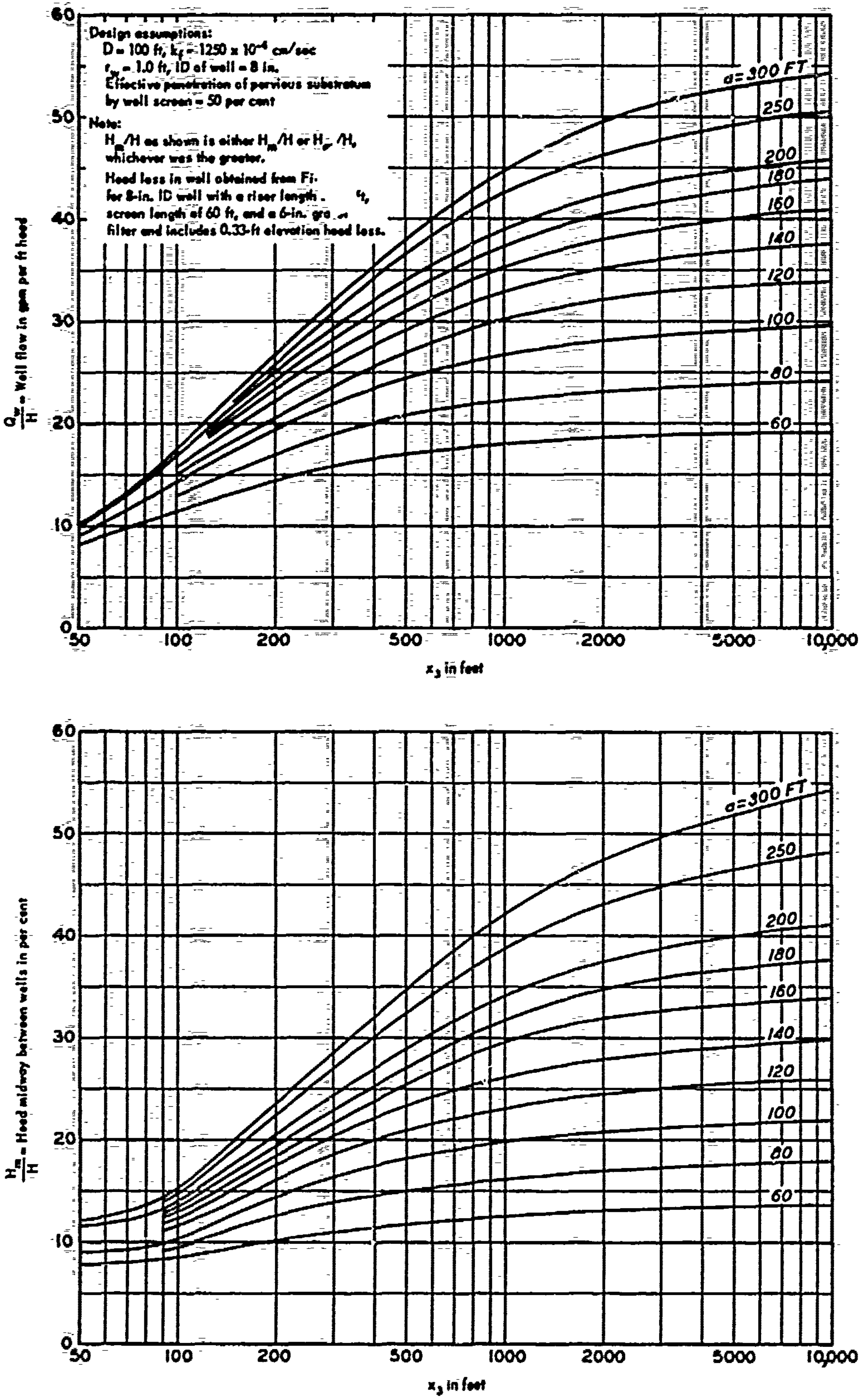


Fig. 63. Well flow and head midway between wells; s = 500 ft

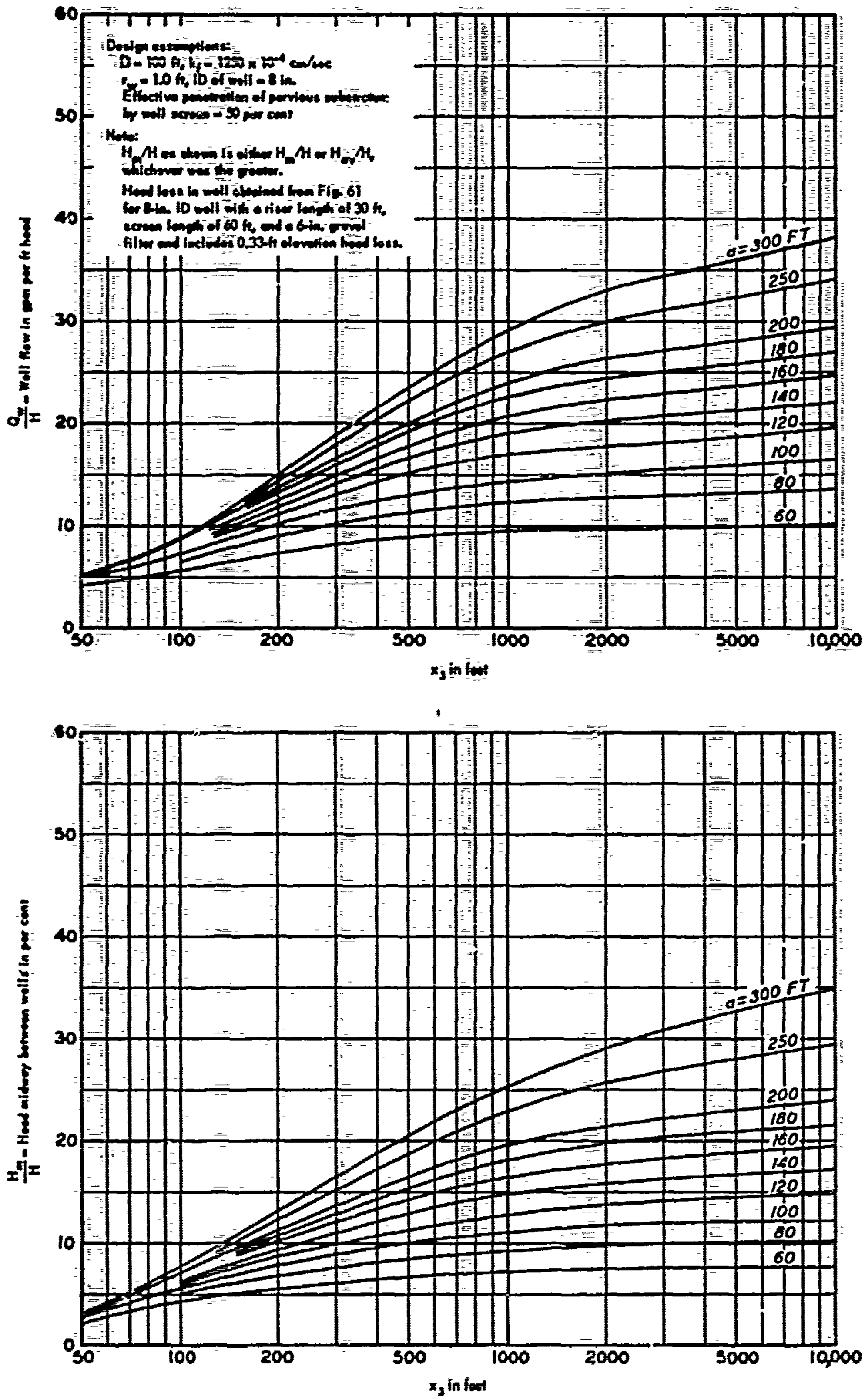
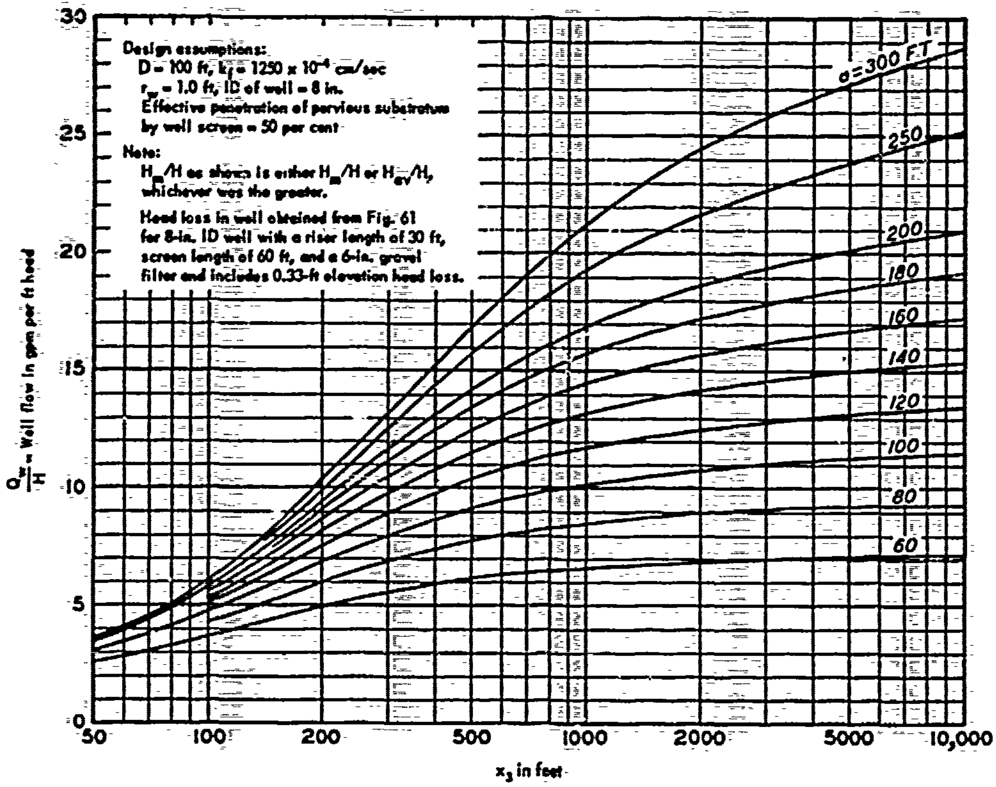


Fig. 64. Well flow and head midway between wells; s = 1000 ft



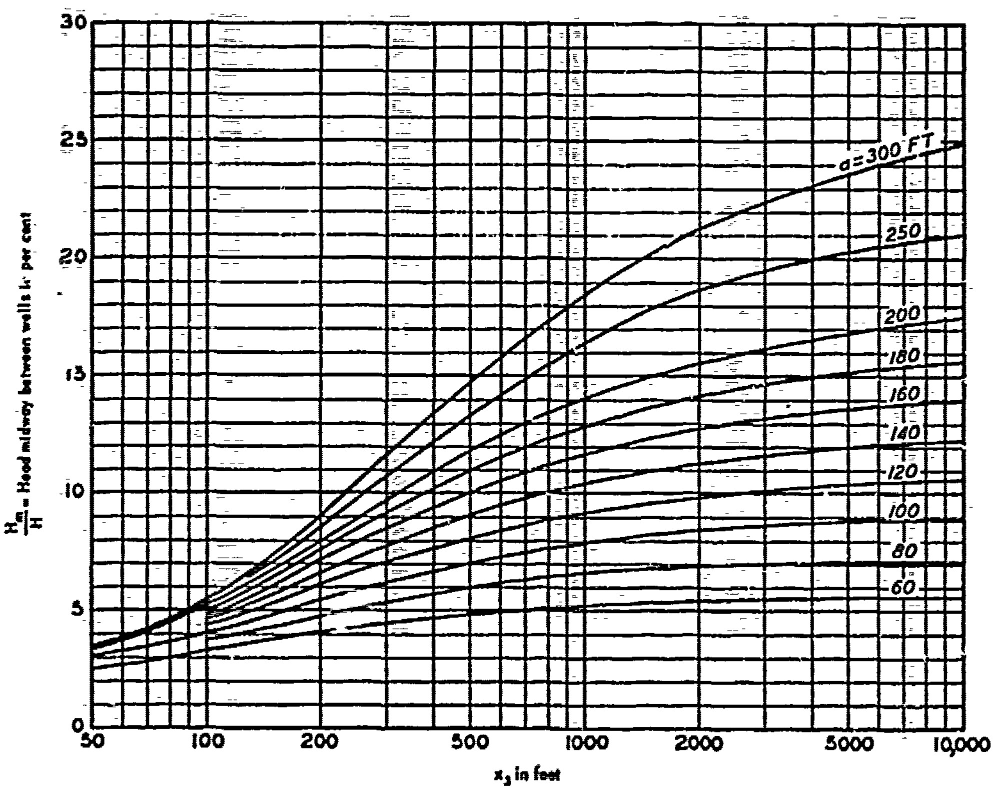
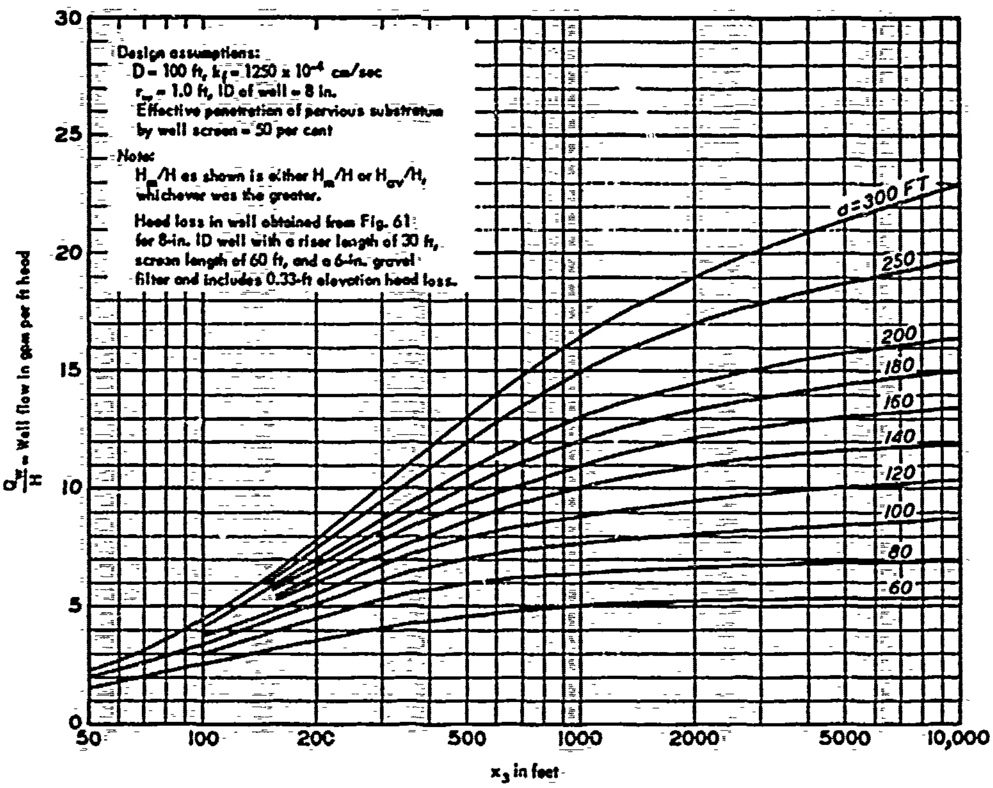


Fig. 65. Well flow and head midway between wells; s = 1500 ft



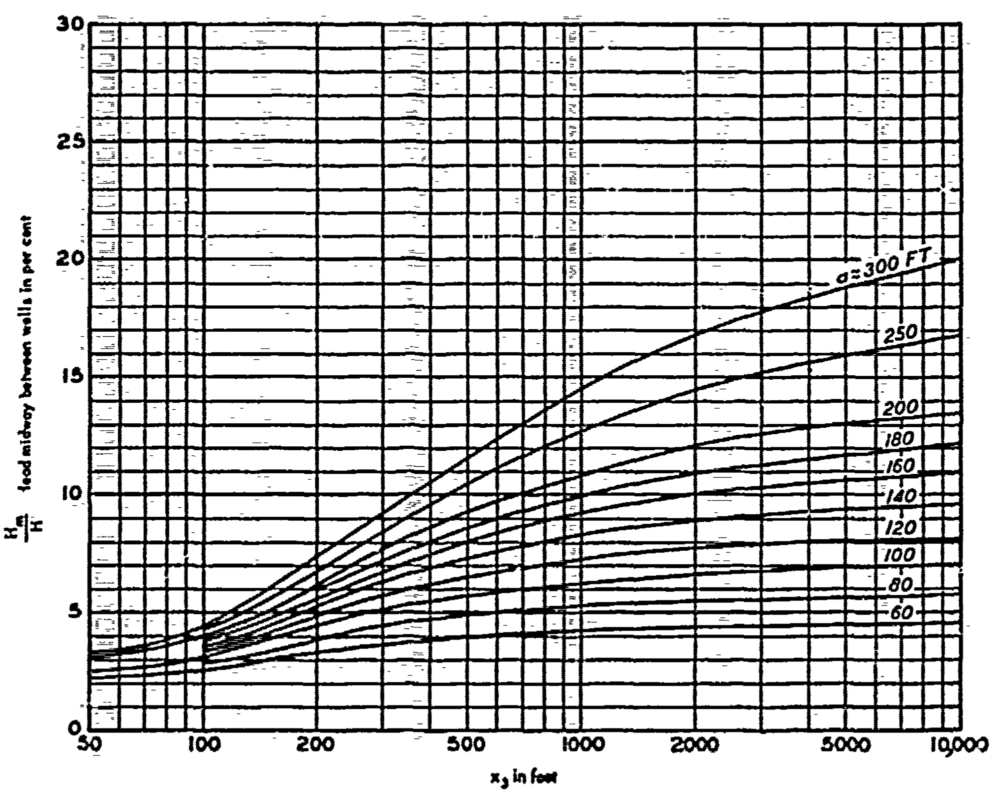


Fig. 66. Well flow and head midway between wells; s = 2000 ft

reaches of levee as regards seepage generally exist upstream of this point. Where k_f is not equal to 1250×10^{-4} cm per sec and/or $D \neq$ 100 ft, Q_w/H can be determined from the curves in figs. 63-66, and then multiplied by $\frac{k_f D}{125,000}$ where k_f is in 10^{-4} cm per sec units and D is in feet. Values of H_m/H and Q_w/H for values of s other than those in figs. 63-66 can be obtained by interpolation.

707. The following numerical example illustrates the use of the design curves. Assume $k_f = 1000 \times 10^{-4}$ cm per sec, $D = 90$ ft, $s =$ 1000 ft, $x_3 = 500$ ft, $H = 24$ ft, $z_t = 8.0$ ft; compute the required spacing of a 750-ft line of 8-in. ID wells penetrating 50% of the aquifer, with $i_o = 0.50$.

$$h_a = i_o z_t = 0.50 \times 8.0 = 4.0 \text{ ft}$$

$$\frac{h_a}{H} = \frac{4.0}{24} = 0.167 \text{ or } 16.7\%$$

From fig. 64, $a = 200$ ft for $H_m/H = 16.7\%$ which is the spacing required for an infinite line of wells. However, for a 750-ft line of wells and $a = 200$ ft

$$\frac{H_{m_N}}{H_{m_\infty}} = 1.12 \qquad \text{from fig. 62}$$

or $\frac{H_m}{H}$ would be 12% greater than the allowable value. Thus the system should be designed assuming $H_m/H = 16.7\% \div 1.12 = 14.9\%$; the required well spacing from fig. 64 is 175 ft. From fig. 64, $Q_w/H = 18.7$ gpm/ft for $a = 175$ ft and $k_f = 1250 \times 10^{-4}$ cm per sec and $D = 100$ ft. For $k_f = 1000 \times 10^{-4}$ cm per sec and $D = 90$ ft.

$$\frac{Q_w}{H} = 18.7 \text{ gpm} \times \frac{1000 \times 10^{-4}}{1250 \times 10^{-4}} \times \frac{90}{100} = 13.5 \text{ gpm/ft}$$

and for $H = 24$ ft

$$Q_w = 13.5 \text{ gpm/ft} \times 24 \text{ ft} = 325 \text{ gpm}$$

It should be noted that the flows from wells near the ends of the system would exceed 325 gpm because of the tendency for seepage to concentrate at the ends of the well line.

708. On the basis of the data shown in fig. 56 for model A, the characteristics of which were similar to those in the above example, it appears that this system of wells would increase the total quantity of seepage passing beneath the levee by about 25%. Computations based on formulas in fig. 59 and data in figs. 60 and 61 indicate that this finite well system would increase the total quantity of seepage by about 20 to 32% as shown by the following computations. The seepage Q_s with no wells would be

$$Q_s = \frac{k_f \, d \, H}{s + x_3} = \frac{0.2 \text{ ft/min} \times 90 \text{ ft} \times 24 \text{ ft}}{1000 \text{ ft} + 500 \text{ ft}} = 0.29 \text{ cfm/ft of levee}$$

or

$$Q_s = 215 \text{ gpm/100 ft of levee} .$$

Well flow per 100 ft of levee, for wells spaced on 175-ft centers, is

$$Q_{w(100)} = 325 \text{ gpm} \times \frac{100}{175} = 185 \text{ gpm/100 ft of levee} .$$

Seepage Q_{sw} beyond the well system can be computed as follows:

$$Q_{sw} = \frac{k_f \, d \, H_{av}}{x_3} .$$

H_{av} can be computed from equation 43 as follows:

$$\frac{D}{a} = \frac{90 \text{ ft}}{175 \text{ ft}} = 0.51, \quad \frac{a}{r_w} = \frac{175 \text{ ft}}{1 \text{ ft}} = 175 .$$

From fig. 60, $\theta_m = 1.10$ and $\theta_{av} = 1.00$. For $Q_w = 325$ gpm, $H_w = 0.8$ ft from fig. 61, and, therefore,

$$h_m = d_m - H_w = 4.0 \text{ ft} - 0.8 \text{ ft} = 3.2 \text{ ft}$$

$$h_{av} = h_m \frac{Q_{av}}{Q_m} = 3.2 \text{ ft} \times \frac{1.00}{1.10} = 2.9 \text{ ft}$$

$$H_{av} = 2.9 \text{ ft} + 0.8 \text{ ft} = 3.7 \text{ ft}$$

thus

$$Q_{sw} = \frac{0.2 \text{ ft/min} \times 90 \text{ ft} \times 3.7 \text{ ft}}{500 \text{ ft}} \times 100 \times 7.5 = 100 \text{ gpm/100 ft of levee}.$$

The percentage increase in total flow due to relief wells is

$$\frac{Q_{w(100)} + Q_{sw} - Q_s}{Q_s} \times 100\%$$

or

$$\frac{185 + 100 - 215}{215} \times 100\% = 32\% .$$

709. It should be noted that the above computations are based on the assumption that the flow without wells would be laminar through the top stratum and that no boils would develop. For conditions in the above example, the maximum possible head at the toe of the levee without wells would be about 0.75 z_t or 6 ft, which corresponds to an H of 18 ft. For H greater than 18 ft, sand boils would probably develop and at an H of 24 ft, Q_s might be as large as 240 gpm, and the per cent increase in total flow as a result of relief wells would be

$$\frac{185 + 100 - 240}{240} \times 100\% = 19\% .$$

710. <u>No landside top stratum.</u> Where no top stratum is present landward of the levee, and it is desired to intercept a certain portion of the seepage beneath the levee by means of relief wells, their spacing on the basis of 50% penetration of the pervious aquifer can be estimated by means of figs. 63-65 as described below.

711. The value of x_3 to be used for an isotropic, homogeneous foundation (either natural or transformed) with no landside top stratum

is equal to 43% of the thickness of the pervious substratum. From model studies, it was found that with no landside top stratum, the increase in total quantity of seepage as a result of the installation of relief wells was only about 5% (see fig. A5). Therefore, the total quantity of seepage passing beneath the levee with or without wells can be estimated from formulas for case 3 shown in fig. 22. The well spacing can then be selected so that the desired amount of seepage will be intercepted.

712. If, for example, $x_1 = 600$ ft, $L_2 = 400$ ft, $s = 1000$ ft, $k_f = 1000 \times 10^{-4}$ cm per sec or 0.2 ft per min, $D = 120$ ft, and $H = 25$ ft, the natural seepage beneath the levee would be 425 gpm per 100 ft of levee from the formula for case 3, fig. 22. If the uncontrolled natural seepage is to be reduced to, say, 200 gpm per 100 ft of levee or by 50%, the required well spacing for 50% penetration wells can be estimated from fig. 64 (for $s = 1000$ ft), using $x_3 = 0.43\ D$ or 52 ft, in the following manner.

713. To intercept 50% of the total natural seepage, the well flow must be about 200 gpm per 100 ft of levee or

$$Q_w \times \frac{100}{a} = 200 \text{ gpm}$$

$$\frac{Q_w}{H} \times \frac{100}{a} = \frac{200 \text{ gpm}}{25 \text{ ft}} = 8.0 \text{ gpm/ft head} .$$

From fig. 64 ($s = 1000$ ft) values of Q_w/H are obtained for various well spacings, a, for $x_3 = 52$ ft, $k_f = 1250 \times 10^{-4}$ cm per sec, and $d = 100$ ft. These values of Q_w/H must then be adjusted to values corresponding to $k_f = 1000 \times 10^{-4}$ cm per sec and $d = 120$ ft.

a in ft	Q_w/H in gpm/ft for $k_f = 1250$, $d = 100$ ft	Q_w/H in gpm/ft for $k_f = 1000$, $d = 120$ ft	$\frac{Q_w}{H} \times \frac{100}{a}$ gpm/ft	Well Flow per 100 ft of Levee in gpm
40	3.1	3.0	7.5	188
50	3.8	3.7	7.4	185
60	4.2	4.0	6.7	168

From this tabulation it appears that the well spacing should be about 40 to 50 ft to intercept 50% of the seepage passing beneath the levee.

714. As well model A-a-3 (Appendix A) was quite similar to the above assumed foundation and seepage conditions, the spacing of wells required to reduce the natural seepage by 50% could be estimated from fig. A5 as 55 ft.

715. Thus, the spacing obtained from model data checks reasonably well that obtained from the above computation. The difference in spacing can be attributed to the fact that the graphs in fig. 64 include head loss in the well, whereas the model data are based on frictionless wells with the top of well at natural ground surface.

Landside Seepage Berms

Use of berms

716. A landside berm can be used to control seepage by increasing the thickness of the top stratum immediately landward of the levee so that the weight of berm plus top stratum is sufficient to resist uplift pressures beneath the top stratum. A properly designed berm will be of such width that the excess head beneath the top stratum at the toe of berm is no longer critical, or the area of possible rupture of the top stratum is removed a sufficient distance from the levee as to no longer endanger it. A landside berm also affords some protection against possible sloughing of the landside slope of the levee as a result of seepage.

717. Berms can be used to control seepage efficiently where the landside top stratum is relatively thin and uniform or where no landside top stratum is present. However, they are not very feasible where the top stratum is relatively thick and high uplift pressures develop as the thickness and width of berm required to reduce upward gradients to those recommended herein would be excessive. Where the landside top stratum is irregular, berms will force the point of seepage emergence farther from the levee, but concentrations of seepage and sand boils may still develop at thin spots in the top stratum at the berm toe. Where a levee is founded on thin top stratum and thick clay deposits lie a short distance

landward of the levee, the seepage berm should be of sufficient width and thickness to cover the near edge of the thick clay if practicable; otherwise, the berm will tend to concentrate the seepage in the area between the berm toe and the thick clays.

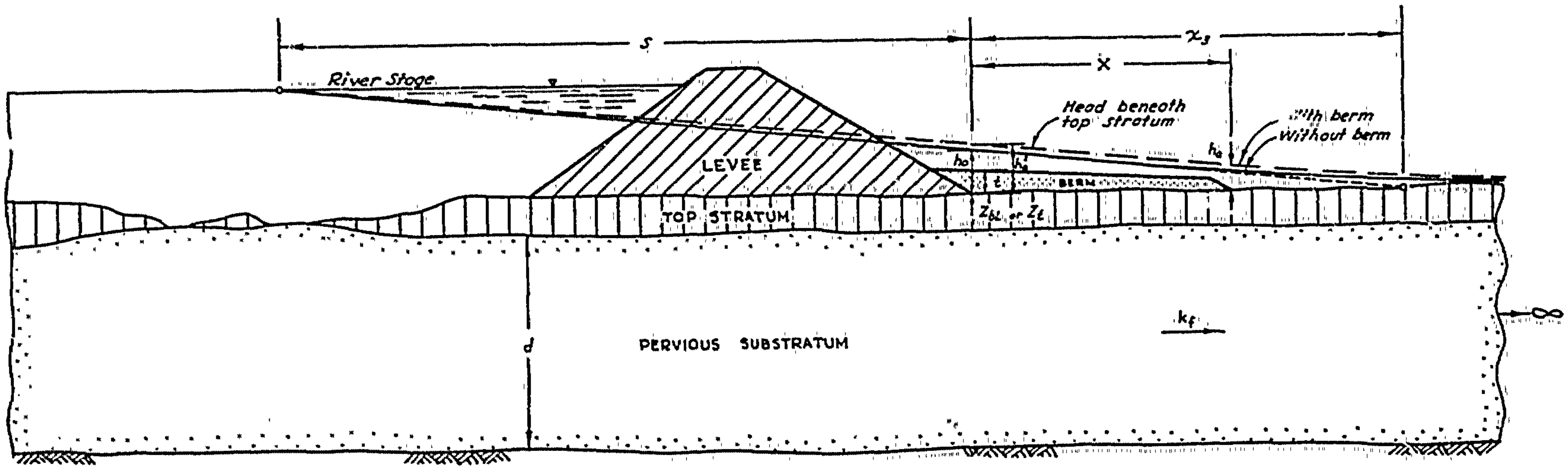
718. Where a levee is founded on a very thin top stratum and is subject to concentration of seepage and the formation of sand boils, the safety of the levee can be improved by adding a landside seepage berm constructed of material borrowed landward of the berm. The near edge of such borrow pits should be about 50 to 100 ft from the berm toe and borrow operations should be controlled so as to insure uniform removal of all of the top stratum down to sand. This will permit seepage to emerge uniformly instead of in the form of sand boils. (The combined base width of levee and seepage berm should provide an adequate creep ratio.) Although this method of seepage control has certain disadvantages, in that it may remove valuable land from cultivation and create undesirable water-filled ponds, it may be better in some situations than removing top strata riverward of a levee for borrow and thereby creating a source of seepage close to the levee.

719. Seepage berms should generally have a slope of 1 on 50 or steeper to insure drainage. However, if the berm is constructed after the levee has caused the foundation to consolidate fully, a slope of 1 on 75 can be used.

Design formulas

720. Berms may vary in character from impervious to completely pervious and free draining. In view of this, design formulas are presented for impervious, semipervious, and sand berms, and a completely pervious, free-draining berm. Where a landside top stratum is present, the berm should have a thickness so that i_o through the top stratum and berm at the levee toe ≤ 0.5 and width if practicable so that the head beneath the top stratum at the berm toe is $0.75\ z_t$ to $0.85\ z_t$. Formulas for designing landside seepage berms overlying a semipervious top stratum are given in fig. 67; items pertinent to the design of each type of berm are discussed below.

721. The formulas shown in fig. 67 permit determination of the



NOTATIONS

$c = \sqrt{\dfrac{k_{bl}}{k_f z_{bl} d}}$

γ_z' = SUBMERGED UNIT WEIGHT OF TOP STRATUM

γ_t' = SUBMERGED UNIT WEIGHT OF BERM

γ_w = UNIT WEIGHT OF WATER

h_o = HEAD AT LANDSIDE TOE OF LEVEE WITHOUT BERM = $\dfrac{Hx_3}{s + x_3}$

h_o' = HEAD AT LANDSIDE TOE OF LEVEE WITH BERM

i_o = ALLOWABLE UPWARD GRADIENT AT LANDSIDE TOE OF LEVEE

i_1 = ALLOWABLE UPWARD GRADIENT AT TOE OF BERM

h_a = ALLOWABLE HEAD AT TOE OF BERM = $i_1 z_t$

X = REQUIRED BERM WIDTH

t = REQUIRED THICKNESS OF BERM AT TOE OF LEVEE

k_t = VERTICAL PERMEABILITY OF BERM

F = FACTOR OF SAFETY AGAINST UPLIFT AT TOE OF LEVEE

Q_s = FLOW INTO BERM PER FT OF LEVEE

FORMULAS

IMPERVIOUS BERM ($k_t = 0$)

$$X_I = x_3\left(\frac{H}{h_a} - 1\right) - s$$

$$h_o' = H\left(\frac{x_3 + X}{s + x_3 + X}\right)$$

$$t = \frac{h_o' - z_t\left(\dfrac{\gamma_z'}{F\gamma_w}\right)}{1 + \dfrac{\gamma_t'}{F\gamma_w}}$$

USE $F \geq 1.5$

SEMIPERVIOUS BERM ($k_t = k_{bl}$)

$$X_{SI} = \frac{-A + \sqrt{A^2 - 24(2 + r)\left(1 + sc - \dfrac{H}{h_a}\right)}}{2c(2 + r)}$$

WHEREIN:

$A = 6 + 3sc\,(r + 1)$

$r = \dfrac{i_o}{i_1}$

$h_o' = h_a\left[1 + cX + \left(\dfrac{2 + r}{6}\right)(cX)^2\right]$

$t = \dfrac{h_o' - i_o z_t}{1 + i_o}$

SAND BERM

$$X_S = \frac{1}{3}\left(X_P + 2X_{SP}\right)$$

$$h_o' = h_a\left[1 + cX + \left(\frac{2 + r}{6}\right)(cX)^2\right]$$

$$t = \frac{h_o' - z_t\left(\dfrac{\gamma_z'}{\gamma_w}\right)}{1 + \dfrac{\gamma_t'}{F\gamma_w}}$$

PERVIOUS BERM WITH COLLECTOR

$$X_P = x_3 \log_e\left(\frac{h_o'}{h_a}\right)$$

$$h_o' = h_o = \frac{Hx_3}{s + x_3}$$

$$t = \frac{h_o' - z_t\left(\dfrac{\gamma_z'}{F\gamma_w}\right)}{1 + \dfrac{\gamma_t'}{F\gamma_w}}$$

$$Q_s = \frac{k_f d H}{s + x_3}\left(1 - e^{\frac{-X}{x_3}}\right)$$

Fig. 67. Nomenclature and formulas for designing landside seepage berms on semipervious top stratum

berm width and the thickness at the toe of the levee; formulas are not given for determining the required thickness at the edge of the crown, as a seepage berm theoretically tapers to zero thickness at its toe. However, it is believed that the thickness of a berm at the crown should be at least 1 ft so as to define the limits of the berm for maintenance purposes. For semipervious and sand berms to function as intended their thickness at the toe of the levee should not greatly exceed the computed thickness. Where landside berms are founded directly on the pervious substratum they should be of such width that the combined width of levee and berm satisfy the creep ratio criteria in table 2. These berms should preferably be constructed of sand, or as a drainage blanket or free-draining berm.

722. Impervious berms. The presence of a landside impervious berm restricts the natural relief of pressure that would result from natural seepage through the top stratum, and thus increases the hydrostatic head at the levee toe with respect to the original ground surface. The effect of an impervious berm on substratum pressures is the same as increasing the impervious base width of the levee a distance equal to the width of the berm. An impervious berm constructed on top of relatively pervious top strata tends to cause the development of relatively large uplift pressures beneath the berm, thereby requiring additional berm thickness. The thickness of the berm at the toe of the levee should be determined from appropriate formula in fig. 67 using a factor of safety of 1.5.

723. Semipervious berm. A semipervious berm is one having a vertical permeability equal to that of the top stratum (see table 38). In this type berm, natural seepage passes through the berm and emerges on its surface. However, even this type of berm will increase the substratum pressure at the levee toe (measured above the ground surface) as the berm creates additional resistance to seepage flow. The required width of a semipervious berm can be computed from the formula in fig. 67. The correctness of this formula has been verified from results of electrical analogy model studies conducted by the Kansas City District, CE. In order for a semipervious berm to function as intended, it must have a permeability equal to or greater than that of the underlying top

stratum and must not be appreciably thicker than the computed thickness. On the basis of values of k_{bL} obtained at the piezometer sites (table 38), it appears that a berm must be constructed of silty sand or fine sand to be classified as semipervious.

724. Sand berm. Sand berms have a slight advantage over semipervious berms in that less berm material is required for the same degree of seepage protection. A sand berm should have a vertical permeability of at least 100×10^{-4} cm per sec. Even with this permeability, seepage into the berm must emerge at its surface, as sand berms do not have sufficient capacity to conduct any appreciable flow landward without excessive internal head loss.

725. Theoretical formulas for design of sand berms were not developed. Instead it was assumed that a sand berm would be more efficient than a semipervious berm but not as efficient as a pervious, free-draining berm (see below), and that the length of a sand berm should be intermediate between that of a semipervious and a pervious free-draining berm. Although a sand berm will be considerably more pervious than a semipervious berm, the presence of a sand berm will increase the landside substratum pressure over that which would exist without a berm, because seepage through the berm must emerge at the berm surface. As a result the dimensions of a sand berm are considered more closely related to those of a semipervious than to those of a free-draining berm, and should have the dimensions given by the formulas for a sand berm in fig. 67.

726. Free-draining berm. A free-draining berm is one where the seepage enters the berm, is collected and discharged landward with low internal head losses in the berm. Such a berm will not affect the natural seepage flow pattern or the distribution of landside substratum pressures and, therefore, is the narrowest and thinnest of all berms required for a given foundation condition. However, for a berm to be free-draining it must be underlain by properly designed sand and gravel filters and a collector system (see Part VII). The sand and gravel blankets beneath the collector pipes should have a minimum thickness of 6 in. The gravel layer should be covered with 4 to 6 in. of the sand filter to prevent the overlying random soil from migrating into the gravel. The landside edge

of the berm should consist of about 3 ft of random soil to protect the gravel blanket against backflooding with muddy surface water. The material above the filter blankets and collector system can be of random soil. The collector system should be capable of collecting and discharging the flow into the berm (which can be estimated from fig. 67) with small head losses. The collector pipes should be of extra-strength vitrified clay tile, or equivalent, perforated with 1/4- or 3/8-in. holes, and should have a minimum ID of 6 in. The ends of the outfall pipes from the collector system should be of unperforated pipe and should terminate in a tee with a short vertical sleeve, rubber gasket and flat-type check valve, and an outlet guard (similar to those shown in figs. 90 and 91) to prevent backflooding with muddy surface water and the entrance of small animals. The discharge of the outfall pipe should be set about 4 to 6 in. above the natural ground surface.

Maximum widths and examples of designs

727. Where the computed width of a berm required to reduce the substratum pressure at its toe to an allowable amount ($\sim 0.8\ z_b$) exceeds 300 to 400 ft, the berm would not be made wider than 300 to 400 ft as it is considered that a levee would probably be safe against underseepage even with sand boils within such distances. Where the selected width of berm is less than the computed width, the head at the toe of the levee or existing berm h'_o would not be as great or t as thick as indicated by the equations in fig. 67. For the selected berms, h'_o would be recomputed assuming an i_1 of 0.8 at the toe of the new berm and a linear piezometric grade line between the toe of the new berm and the point of effective seepage entry. The recommended thickness of the berm would be based on values of h'_o expected to develop with a berm of the selected width, whereas the original computed thickness would be based on the h'_o corresponding to a berm having a width equal to the computed X. The estimated seepage flow Q_s can be determined from the h'_o corresponding to the selected berm.

728. The final selection of a berm should be based on the availability of borrow materials and the relative cost of each type berm.

For comparative purposes, each of the above types of seepage berm has been designed for a typical set of conditions along Mississippi River levees; the results of the designs are shown in table 40. This table illustrates the desirability of utilizing a relatively pervious type of berm for efficient seepage control.

Table 40

Examples of Design of Seepage Berms

Designs based on following conditions:

$H = 25$ ft	$z_{bL} = z_t = 6.0$ ft	$\gamma'_t = 52.5$ lb/cu ft for impervious and semipervious berms
$k_f = 1000 \times 10^{-4}$ cm/sec	$i_o = 0.50$	$\gamma'_t = 57.5$ lb/cu ft for sand berm or pervious berm with collector, $F = 1.6$
$d = 100$ ft	$i_1 = 0.80$	$F = 1.6$ for impervious berm
$k_{bL} = 3 \times 10^{-4}$ cm/sec	$\gamma'_z = 52.5$ lb/cu ft	$L_3 = \infty$
$s = 1000$ ft	$x_3 = 450$ ft	

Type Berm	Required Berm Width X ft	Required Berm Thickness* t, ft	h'_o** ft	Suggested Design Dimensions: Thickness at Berm Crown ft	Suggested Design Dimensions: Berm Width X ft	Suggested Design Dimensions: Berm Slope	Suggested Design Dimensions: Approx Thickness Levee Toe ft	Approx Material Required cu yd per 100 ft of Levee
Impervious	880	7.3	14.2	0.5	800†	1 on 75	11.2	18,200
			10.6	1.0	400	1 on 75	6.3	5,700
Semipervious	280	3.8	8.6	1.0	275	1 on 75	4.7	3,100
Sand	260	3.3	8.3	1.0	250	1 on 75	4.3	2,600
Pervious with collector	215	2.9	7.7	1.0	200	1 on 75	3.7	2,000††

* At toe of levee.
** Head at toe of levee with berm, measured above surface of natural ground.
† Berm width considered longer than necessary. If boils develop 400 ft or farther landward of the toe of the levee, the levee probably would not be endangered. Therefore an alternate design for an impervious berm with a width of 400 ft is also given.
†† Sand and gravel blankets and a collector system are also required.

Drainage Blankets

729. Drainage blankets can be used for control of underseepage where the levee is built on exposed sands and gravels of fairly homogeneous character. However, they are not effective for controlling seepage in deep substrata where impervious strata or even stratified fine sands would exist between the drain and the deeper more pervious sands.

730. Drainage blankets generally are not considered suitable as a means of controlling underseepage along levees in the Mississippi River

Valley because an upper top stratum of clays, silts, or fine sands usually overlies the deeper much more pervious aquifer. However, where the pervious substratum is fairly homogeneous and extends to the surface, drainage blankets properly designed are a satisfactory control measure.

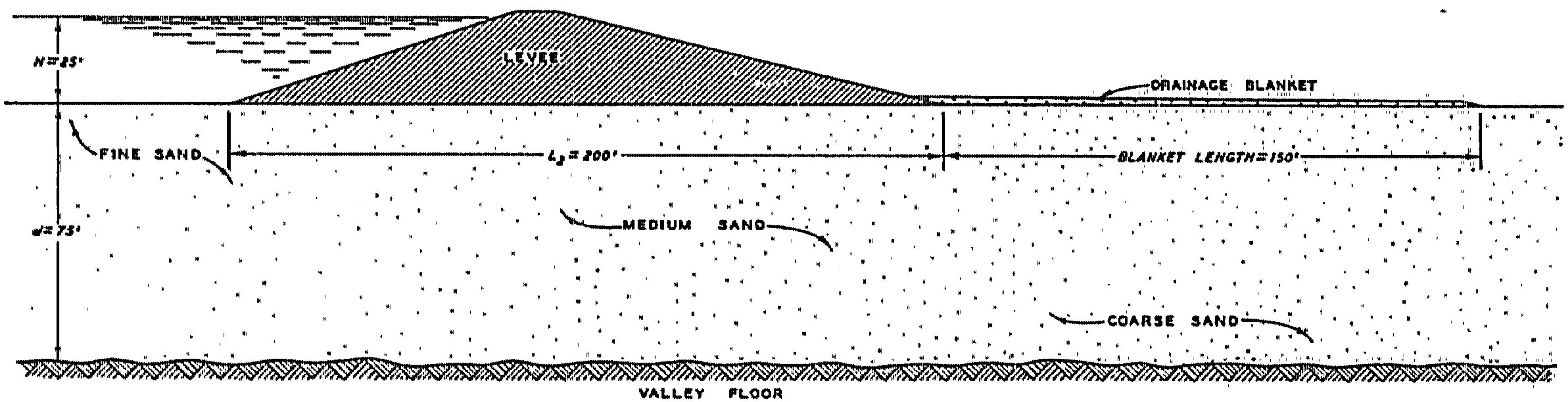
731. A drainage blanket should consist of two or three layers of sand and gravel to provide an inverted filter as illustrated in fig. 68. It should have adequate thickness to prevent piping or a blow-through as a result of local irregularities in the foundation sands, and should be of such length that the concentration of seepage and exit gradient at its toe are not excessive. The filter should be graded to comply with the following filter criteria and each layer should have a minimum thickness of 6 in.

$$\frac{D_{15} \text{ sand blanket}}{D_{85} \text{ foundation sand}} < 5 \ ; \quad 4 < \frac{D_{15} \text{ sand blanket}}{D_{15} \text{ foundation sand}}$$

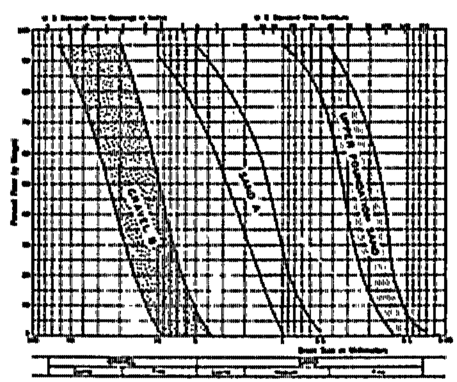
$$\frac{D_{15} \text{ gravel blanket}}{D_{85} \text{ sand blanket}} < 5 \ ; \quad 4 < \frac{D_{15} \text{ gravel blanket}}{D_{15} \text{ sand blanket}}$$

Gradations of typical sand and gravel for drainage blankets complying with the above criteria are shown in fig. 68. In general, a drainage blanket should be about 30 in. thick at the levee toe. Where the levee is founded directly on the pervious substratum, the exit gradient at the toe of the drainage blanket should not exceed 0.8 as estimated from flow net analyses, or the combined base width of levee and blanket should be in accordance with table 2.

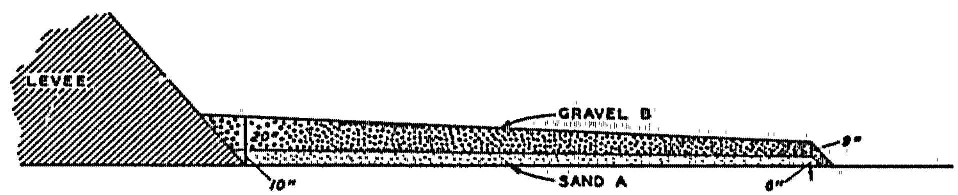
732. An example of the design of a drainage blanket is as follows (also see fig. 68): The length of blanket was determined from table 2 as follows: For the fine upper foundation sand $C = 15$, and assuming no seepage through the blanket, the required base width of levee and blanket = CH = 15 x 25 ft = 375 ft. The width of blanket is $CH - L_2$ or 375 - 200 = 175 ft. However, as natural seepage emerges through the drainage blanket, the actual width of blanket can be less than $CH - L_2$ and a width of 150 ft was selected. The thickness of the blanket shown in fig. 68 is considered to be the minimum necessary for stability of the blanket.



CROSS SECTION OF LEVEE, FOUNDATION, AND DRAINAGE BLANKET



FOUNDATION SAND, AND SAND AND GRAVEL FOR DRAINAGE BLANKET



DETAIL OF DRAINAGE BLANKET

Fig. 68. Example of design of a drainage blanket

Drainage Trenches

733. Drainage trenches can be used to control underseepage where the top stratum is thin and the pervious foundation is shallow so that the trench can be built to substantially penetrate the aquifer. Where the pervious foundation is deep, a drainage trench of any reasonable depth would attract only a small portion of underseepage, its effect would be local, and detrimental underseepage would bypass the trench. Because of the depth of the pervious substratum along Mississippi River levees, drainage trenches are not considered feasible for these levees. However, they may possibly be applicable to levees along the Arkansas River.

734. As in the case of drainage blankets, the existence of moderately impervious strata or even stratified fine sands between the bottom of the drainage trench and the underlying main sand aquifer will render ineffective or decrease the efficiency of a drainage trench. Seepage into a drainage trench, where the top stratum landward of the levee consists of an impervious or relatively impervious blanket, may be computed from the graphs and formulas given in fig. 69. The maximum head landward of the drainage trench may also be computed from these graphs. It is pointed out that the formulas and graphs shown in this figure are applicable only for homogeneous, isotropic, pervious foundations, and for an impervious top stratum landward of the drain. The distance to the source of seepage for these formulas and graphs may be estimated by methods outlined in Part III.

735. If $k_H > k_v$, as is usually the case for alluvial sands, flow to and head landward of a drainage trench can be estimated from fig. 69 after the pervious substratum is transferred to a homogeneous, isotropic formation using $\bar{k}_f$ and $\bar{d}$ for k_f and d, respectively, where $\bar{k}_f = \sqrt{k_H k_v}$ and $\bar{d} = d\sqrt{k_H/k_v}$. A ratio of $k_H/k_v = 4$ is suggested for the Middle and Lower Mississippi River Valley.

736. If the top stratum landward of the drainage trench has a certain degree of perviousness, seepage into the trench, and the maximum head landward of the trench, will be somewhat less than that computed from fig. 69. Therefore, designs based on fig. 69 will be slightly on

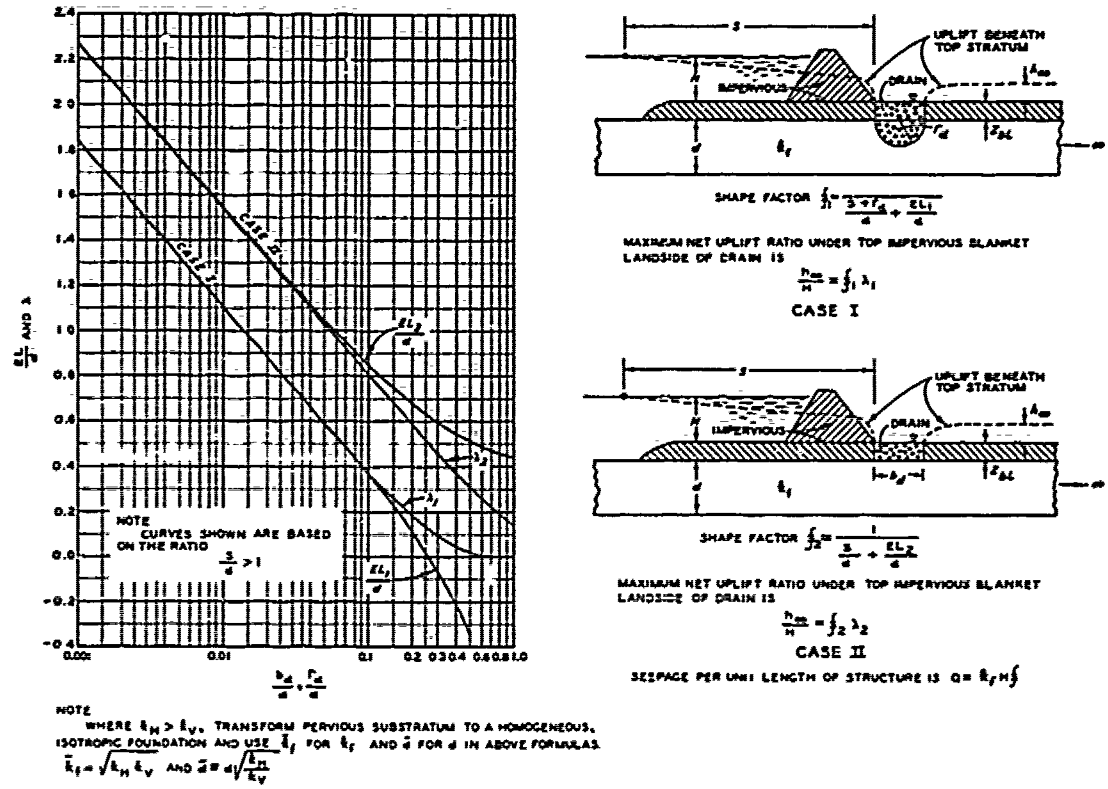


Fig. 69. Formulas and design curves for drainage trenches

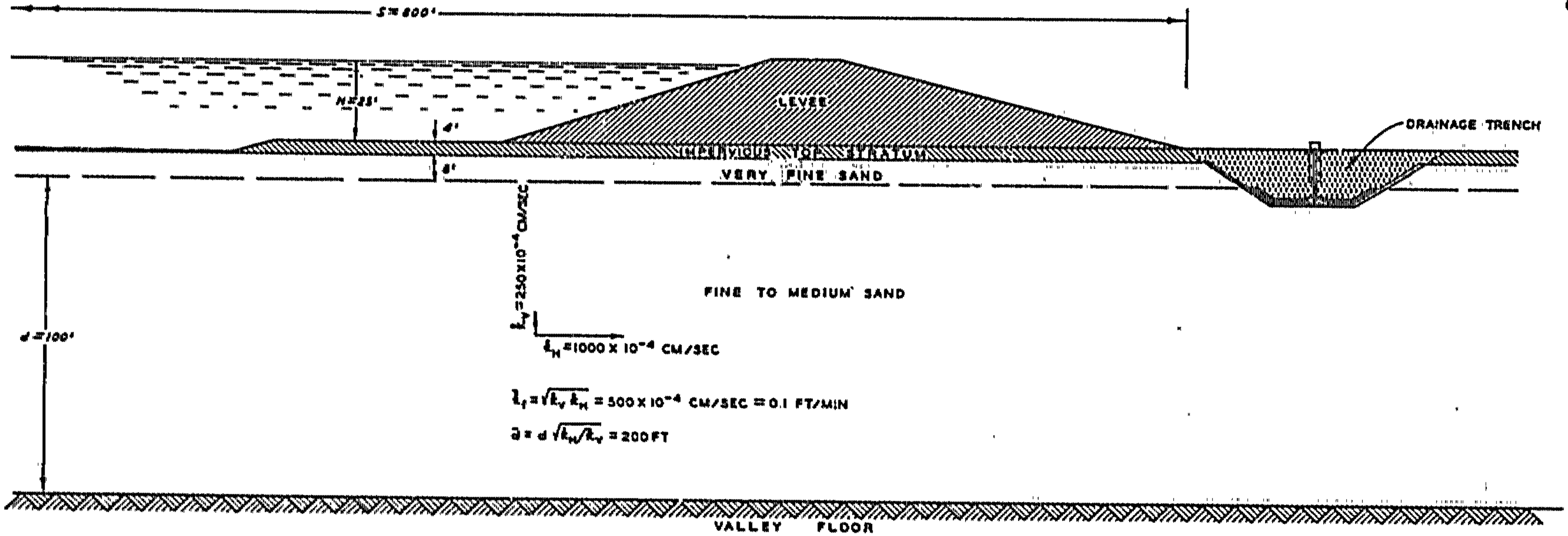
the conservative side if the top stratum landward of the trench is semi-pervious.

737. Where there is no top stratum landward of the levee, seepage flow into the drainage trench and beyond can be estimated from flow net analyses.

738. If the pervious aquifer is highly stratified, or if strata of either clay, silt, or fine sand exist below the bottom of the trench, seepage may bypass the drain and emerge landward of the drain, thereby defeating its purpose. For such cases, other methods of seepage interception are more reliable and efficient. If the trench is underlain by either impervious or semipervious strata, the formulas and graphs shown in fig. 69 are no longer applicable.

739. An example of seepage computations and design of a drainage trench are shown in fig. 70.

740. The filters comprising the drainage layers should be designed



CROSS SECTION OF LEVEE, FOUNDATION, AND DRAINAGE TRENCH

$\frac{z_d}{\partial} = \frac{25'}{200'} = 0.125;\ \frac{EL_2}{\partial} = 0.80$ AND $\lambda_2 = 0.75$

$f = \frac{1}{\frac{s}{\partial} + \frac{EL_2}{\partial}} = \frac{1}{\frac{800}{200} + 0.80} = \frac{1}{4.80} = 0.208$

SEEPAGE FLOW PER UNIT LENGTH OF LEVEE, Q_s:

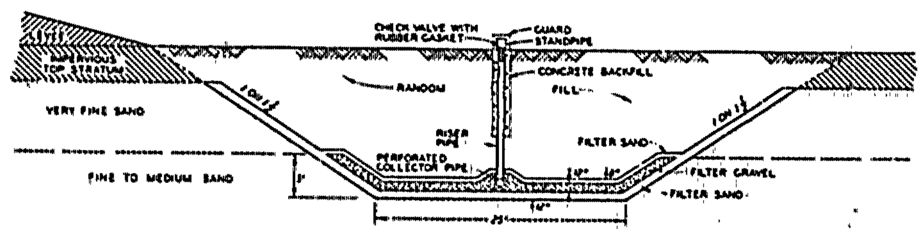
$Q_s = f\,k\,H = 0.208 \times 0.10$ FT/MIN $\times$ 25 FT = 0.52 CFM/FT OF LEVEE

OR 390 GPM/100 FT OF LEVEE

MAXIMUM HEAD BENEATH LANDSIDE BLANKET, h_m:

$\frac{h_m}{H} = f\lambda_2 = 0.208 \times 0.75 = 0.156$ OR 15.6%

$h_m = 0.156 \times H = 0.156 \times 25$ FT = 3.9 FT



DETAILS OF DRAINAGE TRENCH

Fig. 70. Example of design of a drainage trench

in accordance with the filter criteria for drainage blankets. In addition the filter gravel around the perforated collector pipe should comply with the following criteria:

$$\frac{D_{85(min)}\ \text{gravel}}{\text{Perforation}} > \begin{matrix}1.0(\text{holes})\\1.2(\text{slots})\end{matrix}$$

741. The collector pipe for a drainage trench should be made of corrosion-resistant material and should be perforated with 1/4-in. holes. The collector and riser pipes should have adequate capacity to carry the flow to the surface with less than 0.5-ft hydraulic head loss. The head loss should be computed on the basis of maximum full flow through 1/6 the length of collector pipe between risers, the tee connection, and the riser pipe. The riser should be of solid pipe, and should be set about 4 in. above the finished ground surface, as shown in fig. 89 for relief wells. The top of the riser should be provided with a rubber gasket and check valve of the type shown in fig. 91 to prevent flooding of the collector pipe and filters with muddy surface water. The top of the riser pipe may be provided with a low standpipe to prevent flow from the drainage trench at relatively low river stages on the levee. Maximum height of these standpipes should not exceed 1/4 h_c. Of course such standpipes should be removed when they begin to overflow. The top of the riser or outlet should also be protected with a metal guard of the type shown in figs. 89 and 90.

Cutoffs

742. Where practicable, the most positive method of underseepage control is to cut off all seepage beneath a levee by means of an impervious barrier which will eliminate both excess substratum pressures and the problem of seepage water landward of the levee. However, completely cutting off pervious strata 80 to 200 ft deep along extensive reaches of levees is not economically feasible. The installation of partially penetrating cutoffs will not reduce seepage and excess pressures significantly unless the cutoff penetrates 95% or more of the pervious aquifer. However, shallow cutoffs along the riverside toe of levees are feasible where

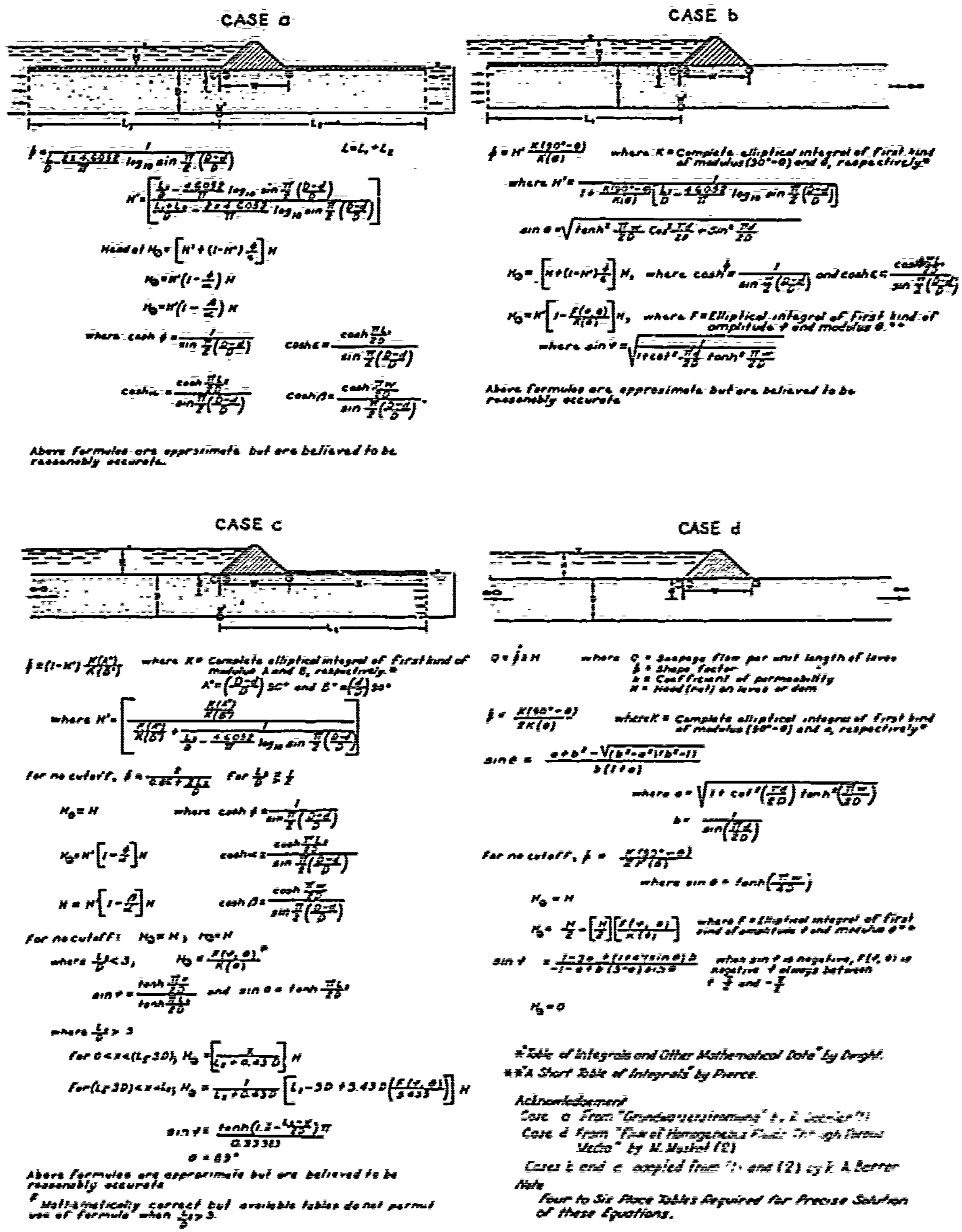


Fig. 71. Formulas for determination of seepage flow and heads for partial cutoffs

necessary to cut off relatively thin layers of either natural levee or crevasse sands which lie immediately beneath the base of the levee and are in turn underlain by more impervious strata.

743. Mathematical formulas for determining seepage flow and heads for partial cutoffs in a homogeneous foundation are given in fig. 71. The hydraulic grade line beneath and landward of a levee underlain by a homogeneous foundation with and without partial cutoffs is illustrated in fig. 72. This top stratum had such characteristics that without any cutoff the excess head at the toe of the levee was 38% of the total head. A 50% cutoff reduced the seepage flow approximately 5% and the excess head from 38% to 37%. Thus it may be seen that partial cutoffs of any practicable depth into a homogeneous aquifer have little effect in reducing seepage flow or substratum pressures landward of a levee.

744. Illustrations of hydrostatic pressure beneath the top stratum with and without cutoffs for a two-layer pervious foundation, a leaking landside top stratum, and two different seepage entrances are shown in

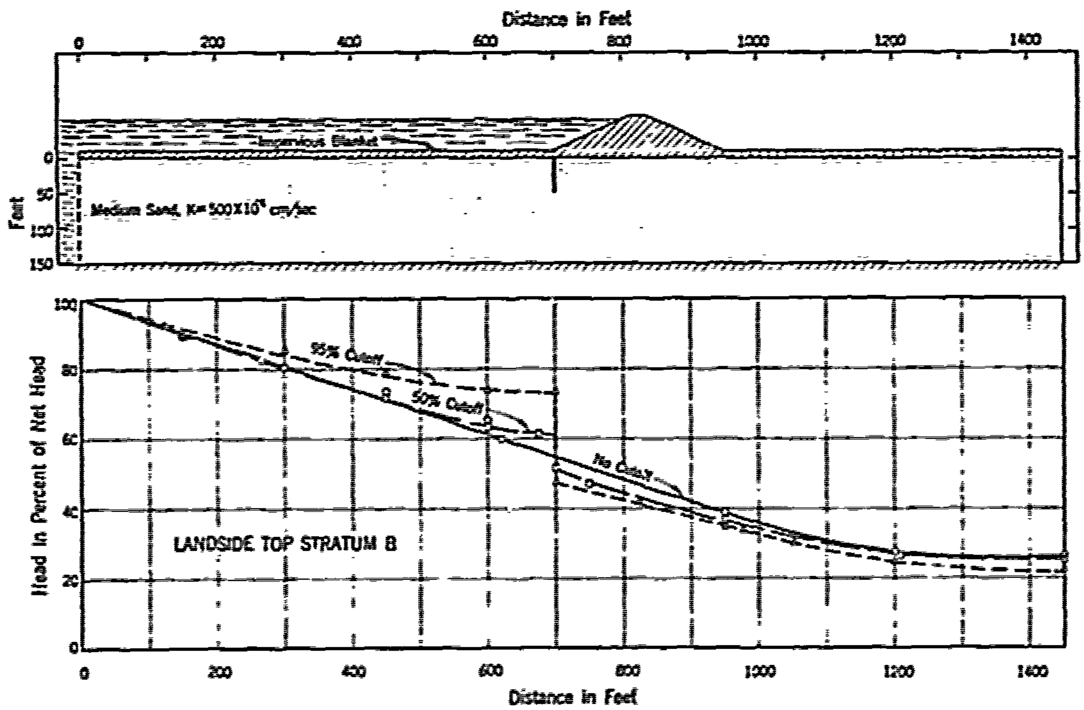
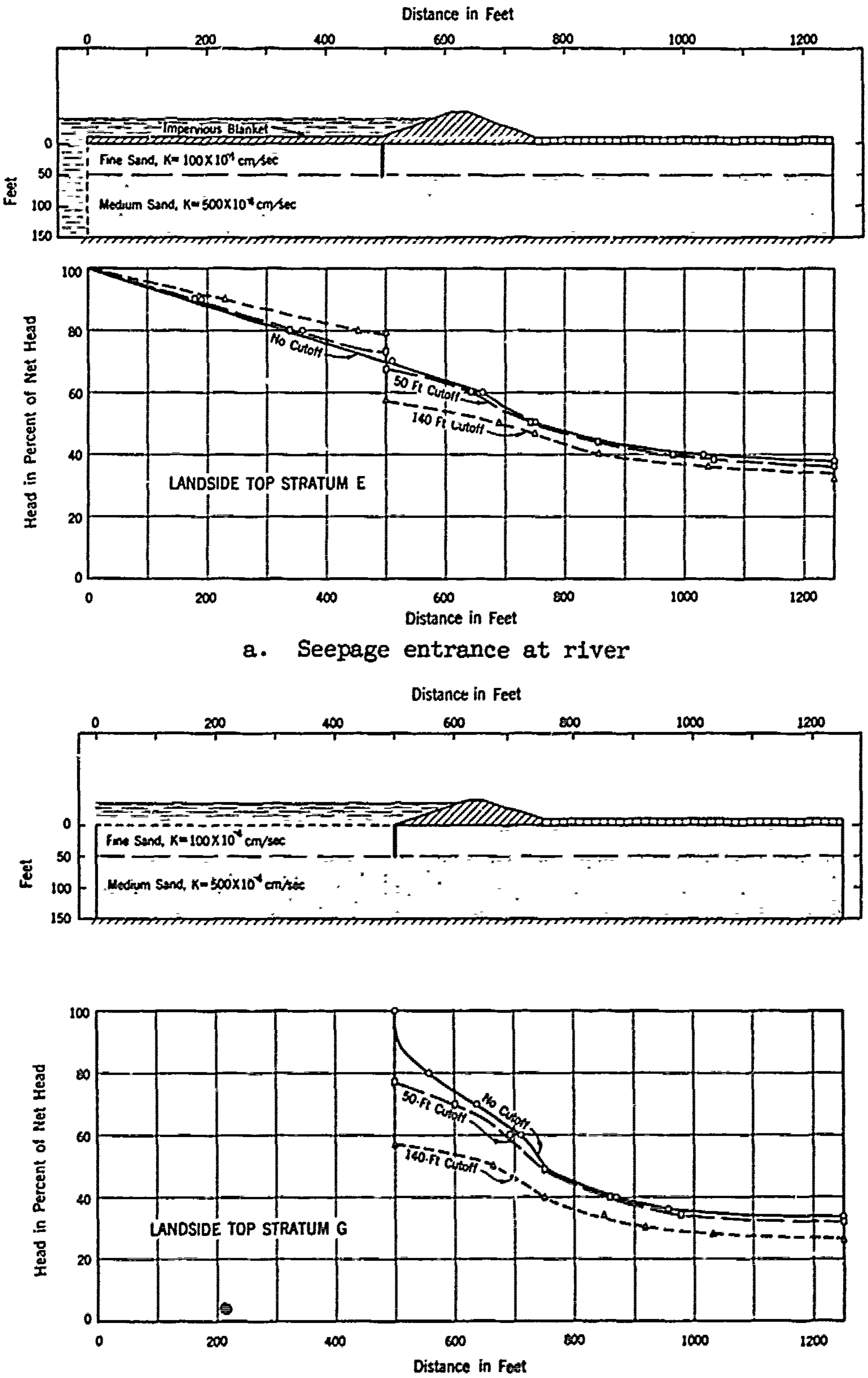


Fig. 72. Hydrostatic head beneath top stratum with various partial cutoffs -- homogeneous foundation



a. Seepage entrance at river

b. Seepage entrance at riverside toe of levee. No riverside top stratum or entrance at river

Fig. 73. Hydrostatic head beneath top stratum with various partial cutoffs -- two-layer foundation

fig. 73. A 50-ft cutoff in this model study reduced the seepage for the first case by 4% and head at the landside toe of the levee from 50 to 49% of that on the levee. In the second illustration in fig. 73 where the seepage riverward of the levee was forced to flow downward through an upper fine stratum, a 50-ft cutoff reduced the seepage flow by only 5%, and the head at the landside toe from 50 to 49% of the head on the levee.

745. Thus, partial cutoffs for controlling seepage beneath levees in the Lower Mississippi River Valley will not significantly reduce the amount of seepage passing beneath the levee or the excess head landward of a levee during high water. Whether or not partial cutoffs would prevent undermining of a levee as a result of piping is not known. If such a pipe developed to within a short distance of a partial riverside cutoff there would be a good possibility that the levee might collapse into the underground cavern and cause a crevasse in spite of the partial cutoff. Fully penetrating cutoffs are not possible along levees in the Middle and Lower Mississippi River Valley because of difficulty of construction and very high cost.

Sublevees

746. A landside sublevee can be used to control seepage by storing water over an area to provide a counterweight against excess head beneath the top stratum in the subleveed area. Sublevees can be used to control seepage where the landside top stratum is relatively thin, and in low areas where seepage normally ponds. A disadvantage of sublevees is that if sand boils occur within the subleveed area, they may be difficult to detect or observe, and may not readily be given emergency treatment, if needed. Control of seepage by sublevees requires proper manipulation of water levels in the sublevee basins during a high water. Controlling underseepage by means of substandard sublevees is potentially hazardous as failure of a sublevee when full of water would result in losing the counterhead at a critical time.

747. A sublevee basin should be of sufficient width to insure that the head at the landside edge of the sublevee is not excessive and the

overflow spillway should be set at such a height that the net excess head at the toe of the levee is not more than the allowable head.

748. The required width X of the sublevee basin can be approximated from the equation for the width of a sand berm in fig. 67. Similarly, the head h'_o at the toe of the levee measured above the ground surface with a sublevee basin can be estimated from the corresponding equation for h'_o with a sand berm of width X. The height of water t in the sublevee basin should be such that

$$t = h'_o - i_o z_t \quad . \tag{50}$$

where $i_o z_t$ is the allowable head at the toe of the levee. The crest of the overflow weir should be at the elevation of the required water surface in the sublevee basin, and should have sufficient length to pass both the design seepage flow Q'_s in cfs into the basin plus runoff from rainfall Q_r. The total discharge Q_T over the weir can be expressed as

$$Q_T = Q'_s + Q_r$$

wherein:

$$Q'_s = \frac{k_f \, d \, L_s \, (H - t/2)}{s + x_3} \left[1 - e^{-\frac{X}{x_3}} \right] \tag{51}$$

$$Q_r = \frac{R \, I \, L_s \, X'}{43560} \text{ in cfs} \; . \tag{52}$$

The sublevee should have a height such that a freeboard of 1 ft will exist above the water surface in the sublevee basin with the overflow weir discharging at a rate of Q_T. The following numerical example illustrates the design of a sublevee basin.

749. Assume the same conditions as those given in table 40 for design of seepage berms. The required width of sublevee basin X will be the same as the required width of the sand berm, or $X = 260$ ft, and the head at the toe of the levee with the sublevee would be (from table 40)

$$h'_o = 8.3 \text{ ft}$$

$$i_o z_t = 0.50 \times 6 \text{ ft} = 3.0 \text{ ft}$$

or

$$t = h'_o - i_o z_t = 8.3 - 3.0 = 5.3 \text{ ft} \quad .$$

Thus the crest of the weir would be set about 5.5 ft above the average ground surface. If the sublevee basin has a length of 500 ft and $X' = 400$ ft, the estimated natural seepage into the basin is

$$Q'_s = \frac{0.2 \text{ ft/min} \times 100 \text{ ft} \times 500 \text{ ft } (25 - 2.8) \text{ ft}}{1000 \text{ ft} + 450 \text{ ft}} \left(1 - e^{-\frac{260}{450}}\right)$$

$$= 67 \text{ cfm/500 ft of levee}$$

or

$$Q'_s = 500 \text{ gpm/500 ft of levee} \quad .$$

Assuming $I = 0.8$ and $R = 3$ in. per hour, the runoff into the sublevee basin will be

$$Q_r = \frac{3 \text{ in./hr} \times 0.8 \times 500 \text{ ft} \times 400 \text{ ft}}{43560} = 1.10 \text{ cfs}$$

or

$$Q_r = 495 \text{ gpm} \quad .$$

Thus the overflow weir should be designed for a discharge of $Q_T = 500 + 495 = 995$ gpm or 2.2 cfs. The net grade of the sublevee should be set 1 ft above the estimated water surface in the basin with the weir discharging at the above rate.

PART VII: INSTALLATION AND CONSTRUCTION OF UNDERSEEPAGE CONTROL MEASURES

Relief Wells

Drilling

750. Relief wells can be installed in a hole made by either the reverse rotary method, the casing method, or other methods that will not remove excess material from the foundation. Methods that involve radical displacement of the formation or may reduce the yield of the well are not permissible. If large cobbles or other obstructions are encountered in the hole for the well and prevent extension of the hole to design depth, the depth of the well may be adjusted or the hole abandoned and another well installed nearby. The holes for all relief wells should be drilled to a depth of 4 ft below that prescribed for the bottom of the well screen.

751. Reverse rotary method. Reverse rotary drilling of holes for wells in sand is basically a suction method in which the material is removed from the hole by a suction pipe. The walls of the hole are supported by seepage forces acting against a thin film of fine-grained soil on the walls, created by maintaining a head of water in the hole several feet above the ground-water table. The film of fine-grained soil is deposited on the walls of the hole by drilling water as the water and soil from the suction pipe are circulated through a sump pit or box in which the sand settles out and from which the water containing the fine-grained particles flows back into the hole.

752. For successful drilling by this method the ground-water table must be about 7 ft or more below the ground surface and there must be an adequate supply of drilling water. This method is excellent for drilling in sand but some trouble may be experienced where silts, sandy silts, and silty sands are encountered.

753. A typical reverse rotary drill unit (fig. 74) normally is equipped with a large-capacity centrifugal pump, 6-in.-diameter drill pipe, and a bit similar in appearance to the cutter head of a dredge (fig. 75). Where cobbles larger than about 3-3/4 in. are encountered, it is

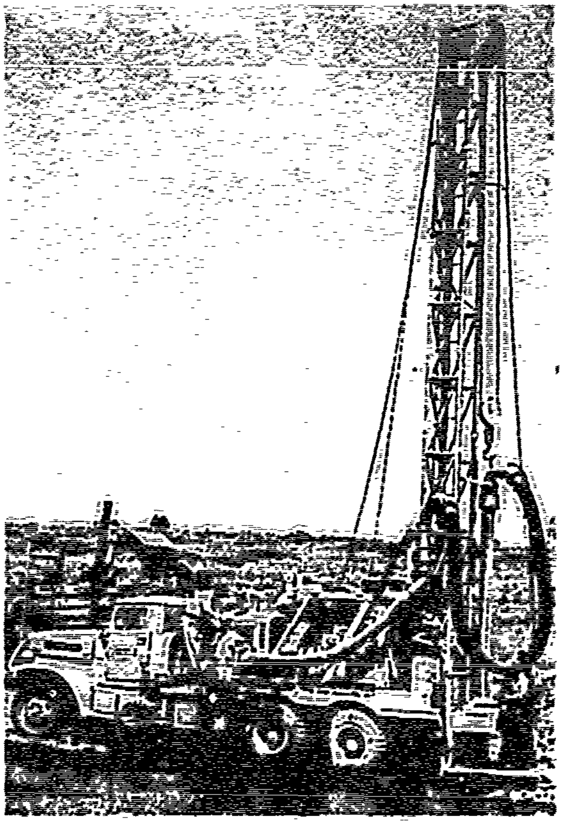
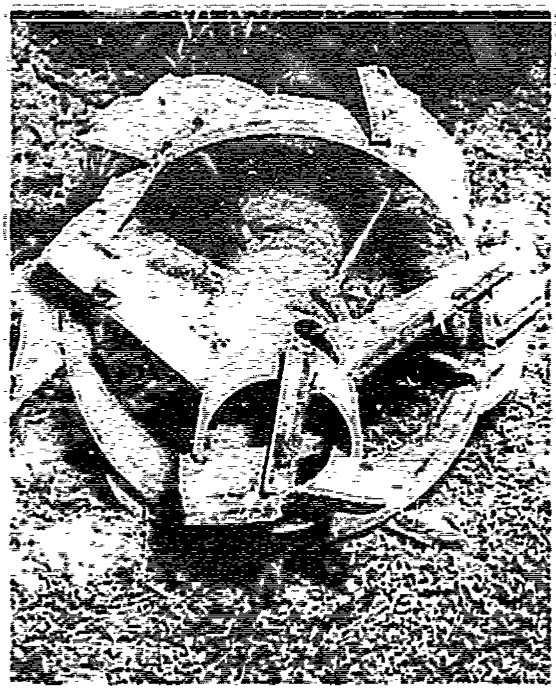


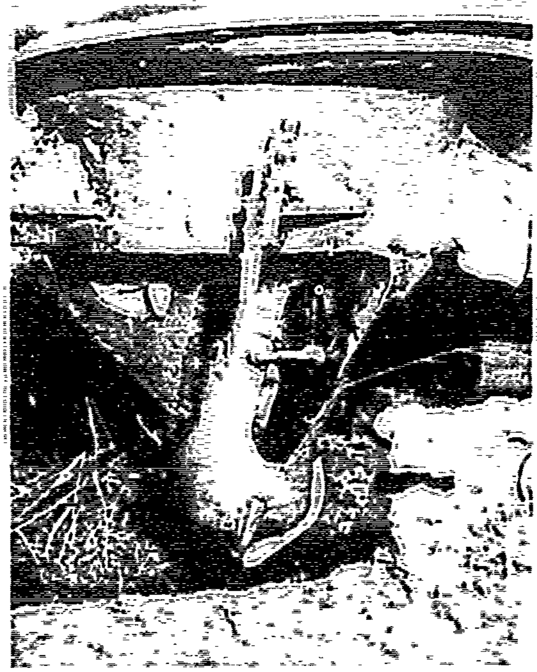
Fig. 74. Reverse rotary drill rig

necessary to provide this equipment with a rock trap. However, holes for relief wells also may be advanced by the reverse rotary method using compressed air or a jet eductor system, neither of which requires use of rock traps.

754. A large sump is required for storing drilling water in the vicinity of each hole. This sump is connected to the hole by means of a ditch of suitable size. At the start of drilling operations, the drill bit is lowered to the ground surface at the well location and the pump is started. Water from the return ditch at the well location is drawn up through the drill pipe, pumped into the sump through a discharge line, and then flows back to the hole through the connecting ditch. After circulation of the drilling water has been started, the drill bit is rotated and lowered as pumping continues. The soil cut loose by the bit is carried up through the drill pipe and pump and is discharged into the sump where the coarse soil particles settle out. The drilling water and finer soil particles continue to be circulated as the boring is advanced. Since the walls of the hole are stabilized by seepage forces resulting from excess hydrostatic pressure, such pressure must be maintained from the time drilling starts until the last of the filter gravel and sand backfill is placed. Should the water supply fail, the well hole will probably cave in and disturb a large volume of foundation soil, and the well will have to be abandoned. It frequently is necessary to use casing through silty or sandy soils encountered near the surface to prevent enlargement of the hole by caving. Caving may also occur when very fine sand or silt is encountered at greater

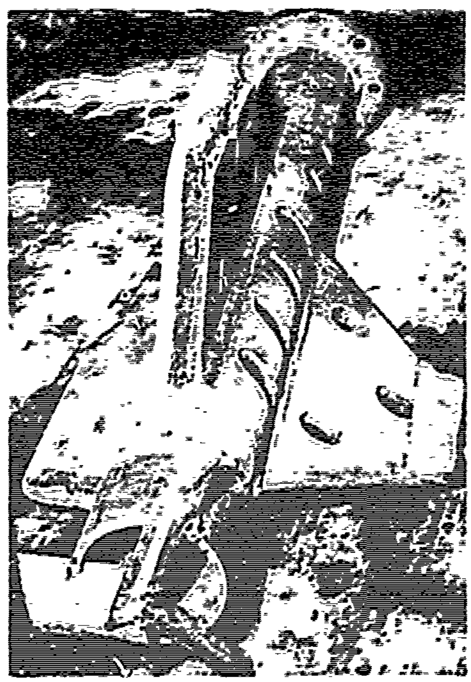


a. General purpose drill bit



b. Drill bit for use in clay

c. Zublin bit for use in shale, cobbles, and sand



d. Drill bit for use in sand

Fig. 75. Reverse rotary drill bits

depths. Such caving can be prevented only by casing the hole or by adding more fines to the drilling water. Commercial drilling muds are available that will stop caving under practically all conditions, but most of them contain bentonite which cannot be successfully removed from the filter; therefore, use of drilling mud containing bentonite is not permissible.

755. Casing method. A temporary well casing may be used to support the sides of the hole during drilling and placement of well screen, riser pipe, and gravel filter. Any casing used should have an ID at least 12 in. greater than the OD of the well screen and should be of a type that can be removed without interfering with the filter or riser pipe. Temporary casing may be set by any approved method that will not create a cavity around the outside of the casing. If the temporary casing becomes unduly distorted, it should be removed and a new casing installed.

Soil sampling during drilling

756. Samples of the foundation soil should be obtained at 2-ft intervals during the drilling of each well. When the reverse rotary method of drilling is used, the samples can be obtained by catching and decanting samples of the effluent from the drill rig in a large bucket or other suitable container. When the wells are drilled by the casing method the samples can be obtained from the bailer or hoisting bucket being used. The purpose of this sampling is to determine the depth at which the screen section of the well can be started and also to determine the existence of strata of silt, silty sands, or very fine sands through which unslotted (blank) sections of pipe should be used rather than slotted screen.

Installation

757. A typical relief well consists of a wood screen section, riser pipe, gravel filter, sand backfill from the top of a gravel filter to an elevation 10 ft below the well outlet, and concrete backfill from the top of the sand backfill to the finished ground surface (fig. 76). (If plastic riser pipe 54 is used, the backfill may consist of compacted impervious soil.) Slotted wooden well screen and riser pipe usually come in lengths of 4, 8, and 16 ft. The screen usually is perforated with slots 3/16 in. (±1/32 in.) wide (fig. 77). The total slot area of 8-in. screen should not be less than 30 sq in. per linear foot of screen. The riser pipe and

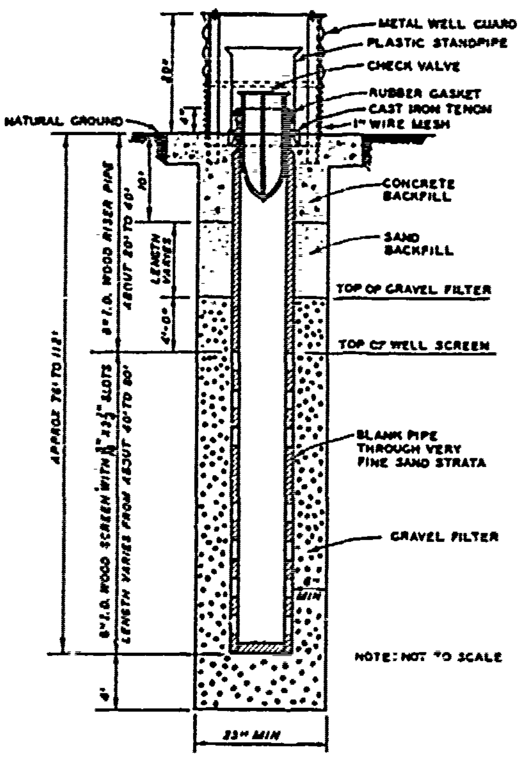


Fig. 76. Relief well and appurtenances

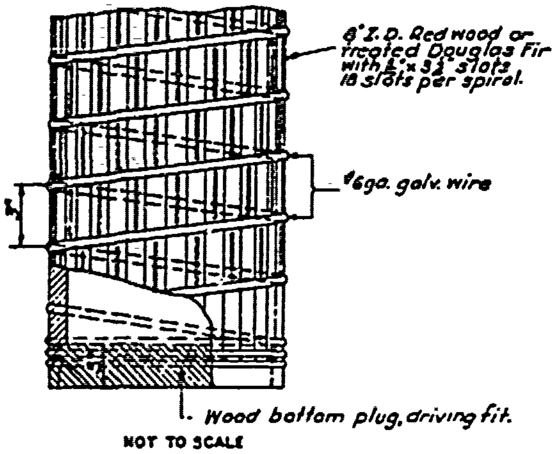


Fig. 77. Detail of wood screen

screen should be constructed of machine-banded wood staves of either untreated redwood, or treated Douglas fir, white pine, or southern yellow pine. The staves should be at least 1-1/8 in. thick and milled from clear, seasoned lumber. Douglas fir, white and southern yellow pine should be pressure-treated with creosote or creosote-coal tar solution with a minimum retention of 8 lb per cu ft for Douglas fir and 10 lb per cu ft for white and yellow pine. The pipe should be banded with heavily galvanized, 6-gage winding wire with 3-in. spacing between wire turns. Wooden plugs are usually used for closing the bottom of the well screens. Bituminous coated corrugated metal pipe may also be used for well screens and riser pipe. The screen portion should be coated after it is perforated. However, wood pipe is considered preferable for relief wells.

758. Length of well screen and depth of well. The length of well screen to be set will depend upon the percentage of effective penetration of the pervious aquifer used in design of the well system. (The principal seepage-carrying aquifer is usually defined as the stratum of medium to fine, or coarser, sand lying above the valley floor.) The top of screen should be set about 4 ft (±2 ft) below the top of medium to fine sand

(D_{50} > No. 50 sieve). Screen should not be placed in silty or very fine sand (D_{60} < No. 70 sieve).

759. The design elevation of the top and the length of well screen are usually estimated from boring data. However, the top of well screen and its length should be checked in the field using data on the soils penetrated as the hole for the well is drilled. The following criteria are suggested where it becomes necessary to modify in the field the lengths of riser pipe, blank pipe, and screen sections in order not to set well screen in strata of very fine sand or finer soil.

a. Minimum length of riser pipe, 16 ft.

b. Maximum depth of well, 125 ft.

c. Screen should not be set in strata of clay, silt, silty sand, or very fine sand. (In setting blank sections of pipe to blank out such strata, an attempt should be made to overlap fine-grained strata with the blank section by 1 or 2 ft.)

d. Riser pipe should extend 2 to 6 ft below soil finer than fine sand.

e. Where lesser footage of blanks is installed than given in the design, the well should still be set to the design depth.

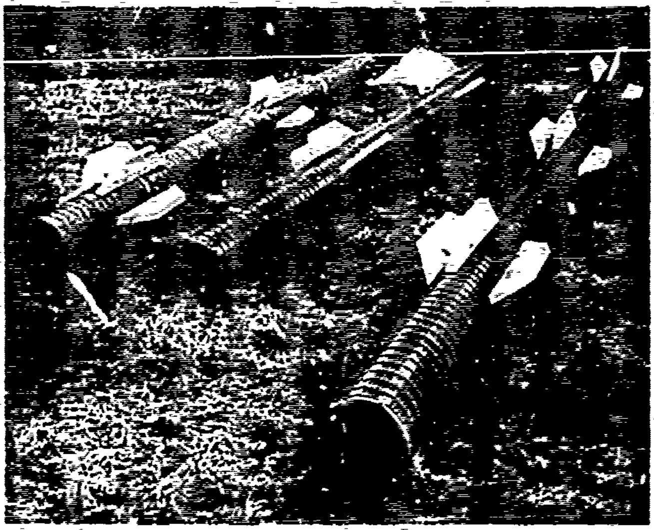
f. Where greater footage of blanks is required than given in design, the well should be deepened in accordance with the following table.

Amount of Blank Additional to That Shown in Design, ft	Amount Well Should Be Deepened ft
4	4
8	8
12	8
16	12
20	12
24	16
28	16

760. Assembly and installation. The well screen and riser pipe should be assembled, or partly assembled, and the bottom of the well screen plugged before the hole for the well is completed. Each joint of pipe and the plug in the bottom of the screen should be fastened securely. Guides should be attached to the assembled riser pipe and screen

in such number and of a type which will center the assembly in the well and hold it securely in position while the filter gravel is placed (fig. 78a). The guides must be of a design that will permit extension of the tremie to the bottom of the hole for the well. (A well can also be centered by using a centering device mounted near the end of a tremie pipe; fig. 78b.) The assembled pipe and screen should be so placed in the hole as to avoid jarring impacts and to insure that the assembly is not damaged nor displaced (fig. 79). The well should be reasonably straight and plumb; a variation of 8 in. in total depth of well from a plumb line at the top of the well is permissible in the alignment of riser and screen. However, adequate clearance for installation of the pumping equipment is required for testing the wells. If any well hole appears to be significantly out of plumb, it should be checked before installing the riser and screen by lowering a 24-in.-diameter, 36-in.-long pipe cylinder to the bottom of the well on a wire rope, checking the inclination of the rope when it is taut and held in the center of the hole at ground surface. A small allowance should be made in setting the top of the well pipe to take care of settlement that may occur during surging and pumping, in order that the top of the well pipe will be at the design elevation.

761. No filter gravel should be placed before the well screen and riser are installed, as the amount required below the screen cannot be predetermined nor placed accurately enough to insure that the top of the riser pipe will be at the correct elevation. Filter gravel should be placed only after the screen and riser pipe have been lowered to the correct depth and the top of the riser pipe has been set slightly above the design elevation. The filter gravel should be placed by lowering the tremie (with narrow slots or small perforations) to the bottom of the hole and then filling it with filter gravel (fig. 80). (The hole for each well should be overdrilled by at least 4 ft to provide space at the bottom for filter gravel which may become segregated when the tremie pipe is first filled.) The tremie should be raised slowly to allow the gravel to run out of the bottom as additional filter material is fed in at the top. The tremie pipe must be kept filled with filter gravel above the



a. Well screen with wooden spiders for centering screen in well

b. Centering device mounted on tremie pipe

Fig. 78. Methods of centering well screen

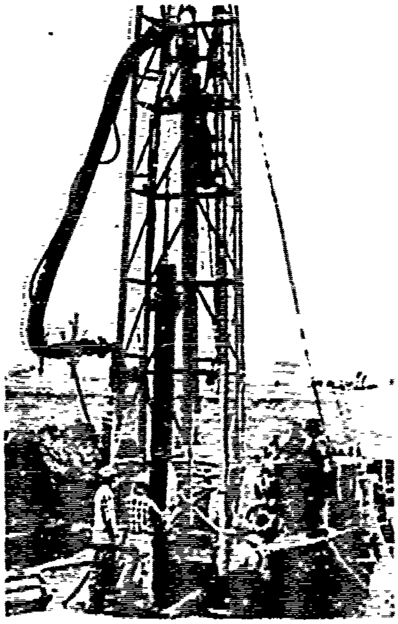


Fig. 79. Installation of well screen

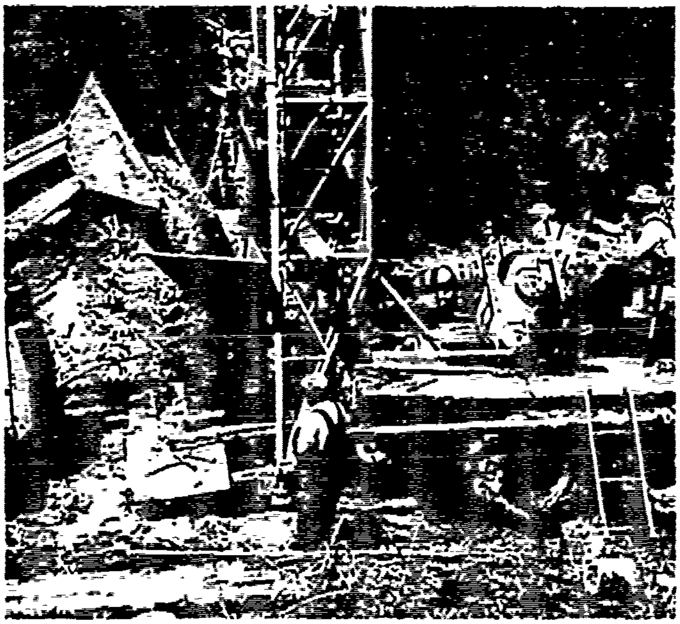


Fig. 80. Placing filter gravel by tremie method

water surface and must be handled at all times in a manner that will prevent segregation of the filter as it is placed. Filter gravel should be placed to an elevation at least 5 ft above the top of well screen to allow for settlement during development of the well.

762. If a temporary casing is used, the filter gravel should be placed in increments not to exceed 2 ft. The tremie and casing should be raised in increments approximately equal to increments of filter gravel placed, except that at no time before completion of the operation should the bottom of the casing be less than 1 ft below the top of the filter. Alternate placing of filter and withdrawing of casing should be continued until the filter has been placed to the desired elevation.

763. Material for the filter around the riser and screen should be a washed sand gravel free from any adherent coating, any vegetable matter, or elongated particles in quantities considered detrimental. In general, the filter should meet the gradation requirements shown in fig. 81, and it should not be skip-graded.

764. Surging and pumping. Materials which may have entered the well during placing of the gravel filter must be carefully removed by pumping or bailing with a piston-type bailer. In some instances, it may be desirable to surge the well slightly before attempting to remove the filter gravel from the well in order to minimize the possibility of

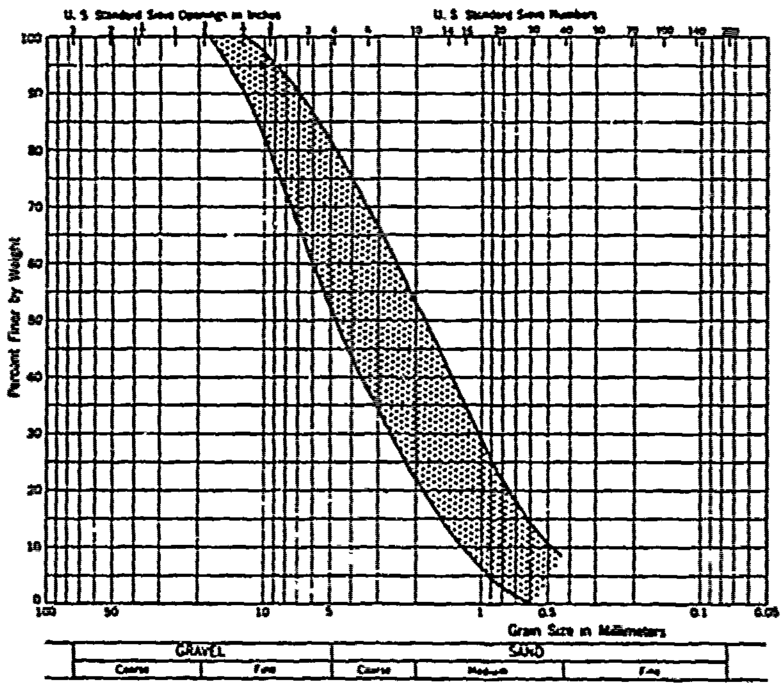


Fig. 81. Gradation of filter gravel for well screen with 3/16-in. slots and alluvial sands in the Mississippi River Valley

trapping the clean-out bailer as a result of filter gravel entering the well above the bailer as it is lowered. After this, the well should be surged and pumped to remove drilling mud or other fines from the filter gravel. Surging and pumping should be started within two hours after the filter is placed and should be continued until the amount of material pulled through the screen between surging operations is less than 0.2 ft in the well, and in any event for not less than 1-1/2 hours. Surging and pumping may be performed alternately or simultaneously.

765. One method for surging is to raise and lower a block made of a heavy rubber disk between two steel disks mounted on a steel rod. The rubber disk should have a diameter approximately 1 in. less than the inside diameter of the well and the steel disks should be 1 in. in diameter smaller than the rubber disk. In some cases it may be desirable to mount two disks approximately 5 ft apart on the steel rod. The surge block should be raised and lowered at a rate of approximately 5 ft per sec. A cycle of surging usually consists of about 20 round trips of the surge block for the full length of the well screen.

766. Another method of surging is to surge and pump the well with air at the same time. This can be accomplished with a device consisting of two disks attached to, and spaced about 4 ft apart on a 3-in. pipe, with perforations in the pipe between the disks and provision made for pumping water out of the pipe by means of air as the surging is done. The lower disk should have a hole in it to permit some flow of water in and out of the space between the disks as the device is moved up and down in the well. The hole will also allow the sand pulled in by the surging operation and not carried out by the rising stream of air and water to fall to the bottom of the well. The pumping can be accomplished by extending a small air pipe down inside the surge pipe to a point between the two disks. The perforations or slots in the surge pipe between the disks will allow water and accumulated foreign material to enter the surge pipe. A small volume of air forced through the small pipe will carry the water and foreign material up to the surface through the surge pipe. In using this procedure for surging and pumping, 5- or 6-ft sections of screen are washed successively until clear water flows and the entire length of screen has been washed.

767. Surging or pumping of such violence as to endanger or damage the well should not be permitted; only sufficient force should be used to remove or loosen all drilling mud and/or fines in the filter gravel and well so that they may be removed by pumping. Some sand may have to be removed by pumping or by a piston-type bailer upon completion of this operation.

768. Within three hours after completion of surging, the well should be pumped to achieve a drawdown of 5 ft in the well or a flow of 200 gpm. If the well produces sand in excess of approximately 2 pt per hour (as determined from sounding and from collection of well flow in a 10-gal container) the well should be resurged and repumped. Wells continuing [illegible] produce excessive amounts of silt or sand after 4 to 8 hours of surging or pumping should be abandoned and properly plugged. After a well is surged and cleaned, it should be kept sealed until provided with a permanent well guard and check valve, to prev[illegible] of surface water.

769. <u>Backfilling.</u> The annular space above the gravel filter should be backfilled after the well has been developed by surging and pumping. Fine or medium sand should be placed to an elevation 10 ft below the well outlet, and concrete should be used as fill to the finished ground surface. When placed in water, the sand backfill should be rodded to insure proper compaction; when placed above water, it should be compacted by approved methods as placement proceeds. Concrete placed in water should be placed by tremie and rodded to insure compaction and absence of voids. If temporary casing is used, it should be withdrawn as backfill is placed so that its bottom is at all times slightly below the top of the placed fill. Sand backfill may be placed prior to surging provided the gravel filter is placed to an elevation 5 ft higher than the top of the well screen. When concrete is placed or cured at temperatures below freezing, the materials should be heated and the finished concrete protected in such manner as will prevent damage because of freezing.

770. If plastic riser pipe is used, it should extend to a depth of about 15 ft below ground surface. Concrete or thoroughly compacted impervious soil may be used as backfill around the pipe.

771. Excess excavated or waste materials should be disposed of so

as to leave the site in an orderly and neat condition. All pits, holes, ditches, and ruts should be filled to original grade with topsoil.

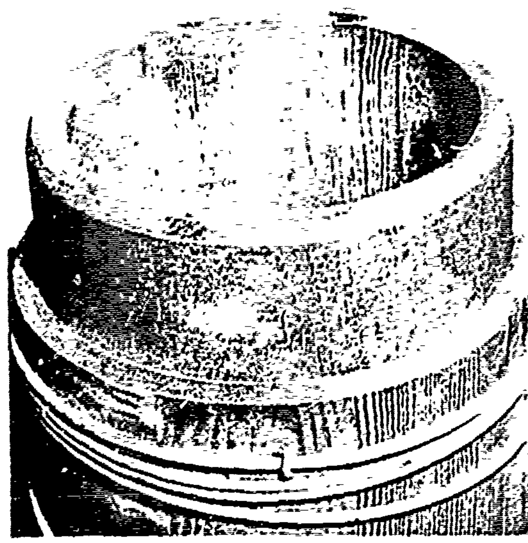
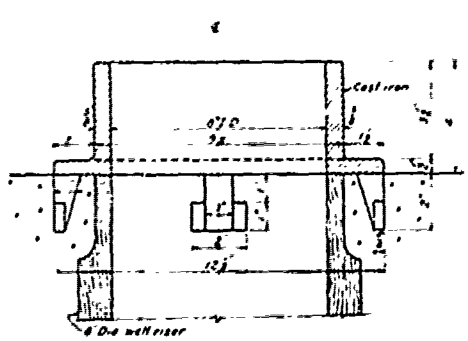
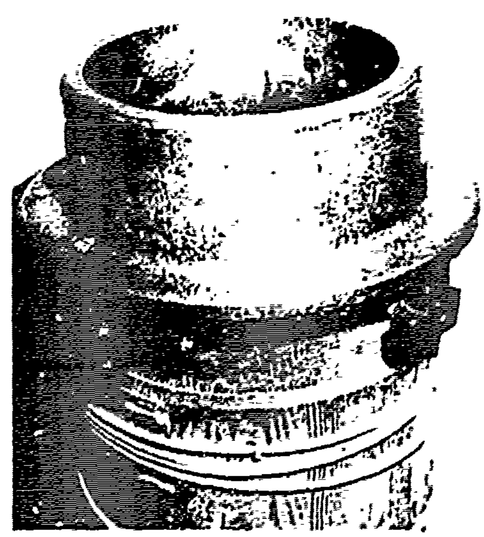


Fig. 82. Tenon end of wood-stave riser pipe

772. Top of well. The concrete around the well is usually finished flush with the normal elevation of the ground. At local high areas in topography, the top of the concrete is sometimes set 0.2 to 1.0 ft below natural ground elevation so that the well outlet can be lowered, thereby improving the efficiency of the well. The top of the riser pipe is usually set about 4 in. above the top of the concrete around the well. The ground immediately around the well should be left flush with the top of the concrete and should be graded to drain to any adjacent lower ground. Although many relief wells have been installed with only the tenon end of the wood riser projecting above the concrete backfill (fig. 82), a more permanent top for a well is a cast iron fitting (fig. 83).



a. Section of cast-iron tenon



b. Cast-iron tenon in place on wood-stave pipe

Fig. 83. Cast-iron tenon for top of relief wells

Pumping tests

773. All wells should be subjected to a pumping test upon completion of installation. If practicable, the pumping test should be made when the water table is at or above the top of the well screen to insure flushing and to check on the stability of the entire length of screen and filter. Equipment for making the pumping tests should include: a pump with a manually operated speed control of adequate capacity; a suction hose or pipe of sufficient length to extend from the pump to a point in the well at least 5 ft below the maximum drawdown required for the test; a discharge hose or pipe of sufficient length to remove the discharge water a satisfactory distance from the well; a suitable sounding device for measuring the depth to water in the well; and a flow meter of standard design for measuring the discharge from the well.

774. The equipment should be set up and the pumping test performed in accordance with the following procedure. The pump should be set near the well and the discharge hose or pipe, with flow meter attached, extended out from the well a sufficient distance to prevent flow back into the well (fig. 84). If an orifice type of meter is used, the section of pipe from the manometer to the orifice should be horizontal and supported rigidly, otherwise the manometer tube on the meter will not remain in a vertical position after the pump is started. The flow meter should be calibrated before using. However, the flow may be estimated for commercial meters from the manufacturer's rating curves. In lieu of an actual calibration or rating curves, the flow from or through

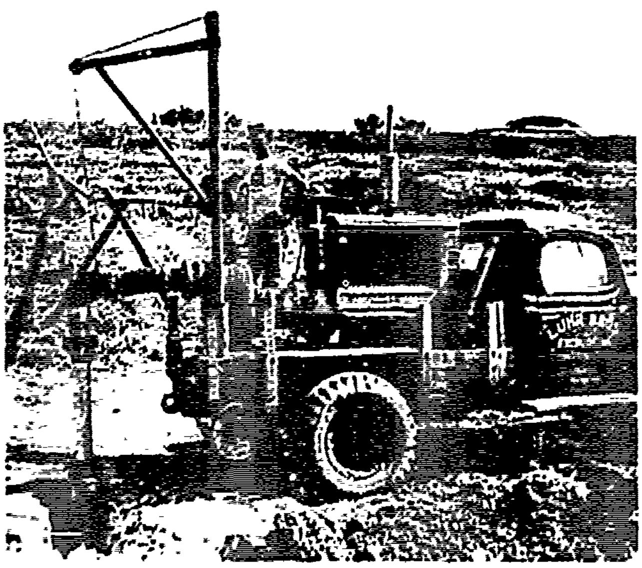


Fig. 84. Pump assembly for test-pumping relief wells

orifices can be estimated from fig. 85.

775. The distance from the top of the well to the water surface should be measured to the nearest 0.01 ft and recorded after the test equipment is set up. The pump then should be operated so as to create a drawdown in the well of 5 to 8 ft or a flow of 500 gpm. The flow from the well should then be recorded together with the time and measured depth (nearest 0.01 ft) to the water surface in the well. For the remainder of the pumping period the flow from the well should be maintained at a relatively constant rate, varying the pump speed if necessary. The flow from the well and depth to water surface in the well should be recorded at 15-min intervals for the remainder of the test period. Should radical variations in pumping rate or interruptions in pumping occur, the pumping test should be started over.

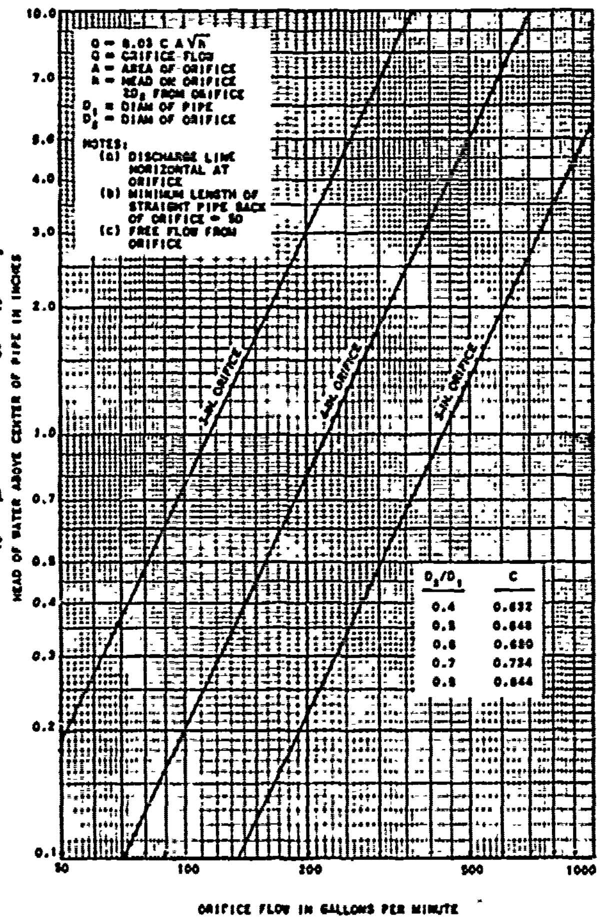
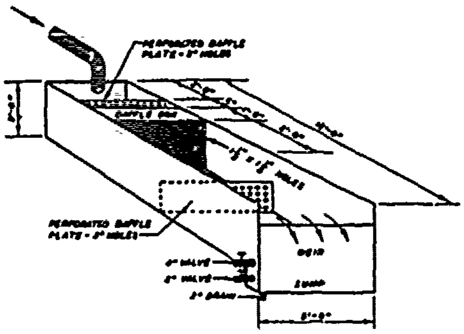


Fig. 85. Flow from orifices at end of 6-in. pipe

776. Pumping tests may be made as soon as a well is completed. Care should be taken not to damage either the top of the well or metal well guard. The period of pumping, discharge rate, and drawdown to be achieved should be in accordance with specifications for the project. No test pumping of a well should be permitted concurrently with drilling, surging, or pumping of any other well within 400 ft of the well being tested.

777. When it is desired to check the inflow of sand during a pumping test, the flow from the well should be discharged into a suitably



a. Diagram of tank

b. Tank in use

Fig. 86. Sand collection tank for checking stability of wells

baffled stilling tank (fig. 86) to cause any fine sand in the water to settle out. The inflow of sand into a well during a pumping test can be determined in the following manner:

<u>a</u>. Determine the depth to top of sand in the well to the nearest 0.01 ft with a flat-bottomed sounding device and steel tape before starting the pumping test.

<u>b</u>. At 15-min intervals during pumping, determine the exact depth (nearest 0.01 ft) to sand in the well and amount of sand collected in stilling tank (to nearest 0.1 pt).

<u>c</u>. Total inflow of sand into the well should be computed for each time interval as the test progresses. The pumping test should be continued until the rate of inflow of sand is less than 1 pt per hour but for no longer than 8 hours. If the rate of sand inflow is more than 2 pt per hour after 8 hours pumping, the well should be abandoned or repaired if practicable.

778. In the event sand or other material collects in the well as a result of the pumping test, it should be removed.

<u>Recording of data</u>

779. Installation data should be recorded on a form such as shown in fig. 87. Any unusual conditions encountered or instances of noncompliance with specifications by the contractor should also be entered on the form. All items in fig. 87 are self-explanatory with the exception of the following.

RELIEF WELL INSTALLATION REPORT

PROJECT: St. Louis District, Underseepage	LEVEE DISTRICT: Prairie du Rocher

LOCATION (STA): 419 + 90	OFFSET FROM CENTER LINE: 128 ft	WELL NO.: 45

DRILLING METHOD: Reverse - Rotary	DATE AND TIME:– From: 17 June '53 4:30 PM To: 18 June '53 5:00 A.M.

WELL SCREEN:– Type: Wood stave, machine banded; Inside Diam: 8-in.; Perforations: 3/16 x 3 1/4-in. slots

RISER:– Type: Wood-stave pipe, machine banded; Inside Diam: 8-in.	FILTER: Gravel
TYPE OF WELL GUARD: Galvanized corrugated metal pipe with wire screen	TYPE OF BACKFILL: Concrete

CONTRACTOR: Luhr Bros. Inc.	CONTRACT NO.: 4846	FOREMAN: Riechmann OPERATOR: Dees
GROUND ELEV AT WELL: 378.2	DEPTH TO:– Fine Sand: 27.6 ft	Medium To Fine Sand: 37.3 ft
ELEV TOP OF RISER: 378.6	ELEV TOP OF WELL GUARD: 380.0	ELEV TOP OF CONCRETE BACKFILL: 378.3

TYPE OF CHECK VALVE: Aluminum plate	WELL EXTENSION:– Type: None; Ht:

FINAL WELL INSTALLATION DATA

RISER PIPE LENGTH: 31.1	INSIDE DEPTH OF WELL: 77.65
"EXTRA" SCREEN LENGTH: 0.0	DEPTH TO SAND IN WELL AFTER CLEANING: 77.58
"DESIGN" SCREEN LENGTH: 39.1	ELEV TOP OF WELL SCREEN: 347.5
TOTAL SCREEN LENGTH: 39.1	ELEV TOP OF FILTER: 368.8
"BLANK" PIPE LENGTH: 7.7	INSIDE BOTTOM ELEV OF WELL: 300.95
DEPTH OF HOLE: 79.3	ELEV RISER SET AT: 378.7

SURGING DATA

METHOD: Block and bailer	DATE AND TIME: 18 June '53 6:05 AM	TOTAL SURGING TIME: 1:00 hr

GRAVEL IN WELL BEFORE SURGING: 15.15 ft	MATERIAL SURGED IN:– [SURGING CYCLE]	#1: 20.65	#2: 5.85	#3: 1.75

[SURGING CYCLE]	#4: 0.65	#5:	#6:	#7:	#8:	Total: 44.05 ft

SETTLEMENT OF FILTER DURING SURGING: 1.5 ft

PUMPING DATA

RATE OF PUMPING: 200 GPM	TOTAL PUMPING TIME: 2 hrs
RATE OF SAND INFILTRATION:– Start: 0.5 pts/hr	End: 0.0 pts/hr

REMARKS: Design depth of well = 84 ft. Cobbles encountered at depth of 79.3 ft. Unable to obtain design depth.

LOG OF HOLE	DEPTH	PIPE INSTALLED
	0	
	10	Riser
	20	
F Sand	30	31.1
MF Sand	40	Screen
M Sand	50	54.8
Clay	60	Blank 62.5
FM Sand	70	Screen
M Sand Cobbles	80	77.65
	90	
	100	

Wayne A. Smith
INSPECTOR

Fig. 87. Form for recording relief well installation data

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RELIEF WELL PUMPING TEST REPORT

PROJECT: St. Louis District, Underseepage	LEVEE DISTRICT: Prairie du Rocher
LOCATION (STA): 419 + 90	ELEV TOP OF RISER: 378.60 — WELL NO.: 45
DATE: 1 July 1953	TIME TEST STARTED: 12:20 PM — TIME TEST COMPLETED: 2:35 PM

TIME PM	ELAPSED TIME MINUTES	DEPTH TO WATER	DRAWDOWN IN FEET	FLOW IN GPM	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER	DRAWDOWN IN FEET	FLOW IN GPM
12:20	0	15.35	0	0	2:00	100	21.57	6.22	500
12:30	10	21.32	5.97	505	2:15	115	21.58	6.23	500
12:45	25	21.46	6.11	500	2:30	130	21.57	6.22	500
1:00	40	21.50	6.15	500	2:35	135	15.66		
1:15	55	21.47	6.12	500					
1:30	70	21.52	6.17	495					
1:45	85	21.53	6.18	500					

SAND INFILTRATION TEST

DEPTH OF WELL: 77.65 — DEPTH TO SAND IN WELL BEFORE TEST: 77.58 — SAND IN WELL BEFORE TEST: 0.07 ft

TEST NO.	TIME	DEPTH TO SAND (Ft)	SAND IN WELL (Pts)	GAIN OR LOSS OF SAND IN WELL (Pts)	SAND PUMPED OUT OF WELL (Pts)	TOTAL INFLOW OF SAND INTO WELL (Pts)	LENGTH OF TEST (Min)	RATE OF SAND INFILTRATION (Pts/Hr)
1	12:30	77.58	1.54	+0.22	0.20	0.42	30	0.84
	1:00	77.57	1.76					
2	1:15	77.57	1.76	0.00	0.10	0.10	15	0.40
	1:30	77.57	1.76					
3	1:45	77.57	1.76	0.00	Trace	Trace	15	Trace
	2:00	77.57	1.76					

DEPTH TO SAND IN WELL AFTER TEST: 77.57 — SAND IN WELL AFTER TEST: 0.08 ft — SAND IN WELL AFTER CLEANING: 0.00 ft

REMARKS:

R. L. Showmaker
INSPECTOR

Fig. 88. Form for recording relief well pumping test data

a. The first item in line 9, "Elev Top of Riser" is to designate the final elevation of top of riser as established after completion of the well.

b. "Elev Riser Set at" as shown in the second column under "Final Well Installation Data" is to designate the elevation at which the top of riser was set initially. (Field experience will indicate the elevation for initial set of top of riser in order that final elevation will be as near that required as possible.)

c. "Inside Bottom Elev of Well" is the elevation of the top of the plug in the bottom of the well screen and is equal to "Elev Top of Riser" minus "Inside Depth of Well."

Pumping test data should be recorded on a form such as shown in fig. 88.

Appurtenances

780. The tops of all wells should be protected with a suitable guard such as shown in figs. 89 and 90. Use of these devices will greatly reduce required maintenance.

781. Each well should be protected against backflow of surface water by a rubber gasket and check valve such as shown in fig. 91. Accelerated full-scale laboratory tests have indicated this check valve to be very effective in preventing backflow of water into a well under adverse conditions. The rubber gasket shown in fig. 91 was specially

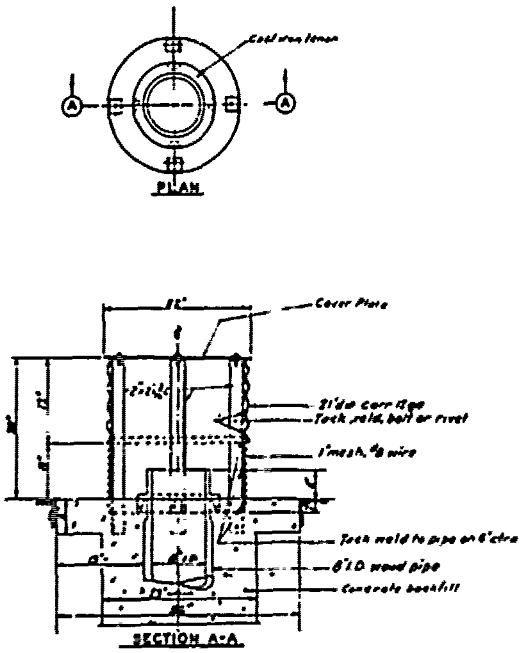


Fig. 89. Metal well guard

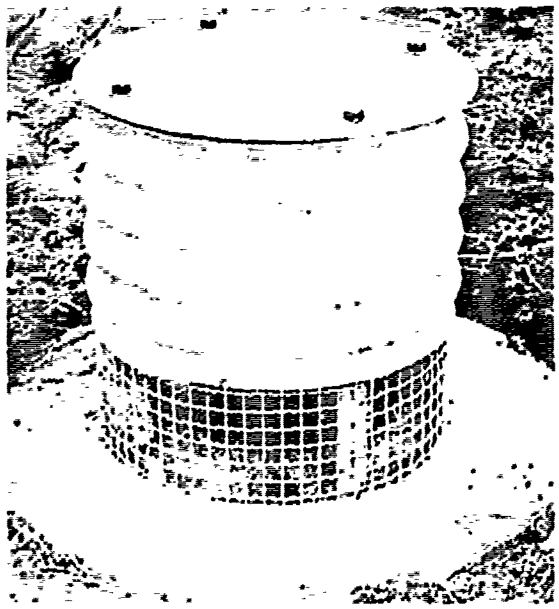
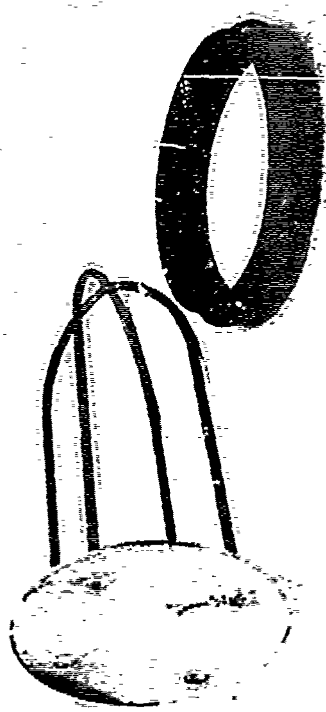
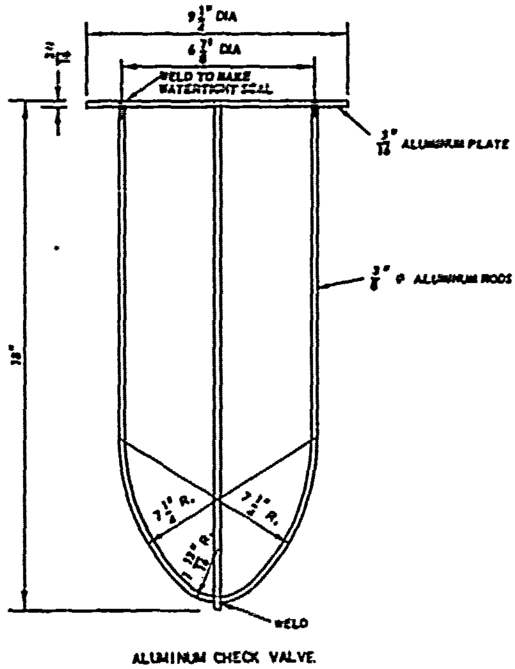


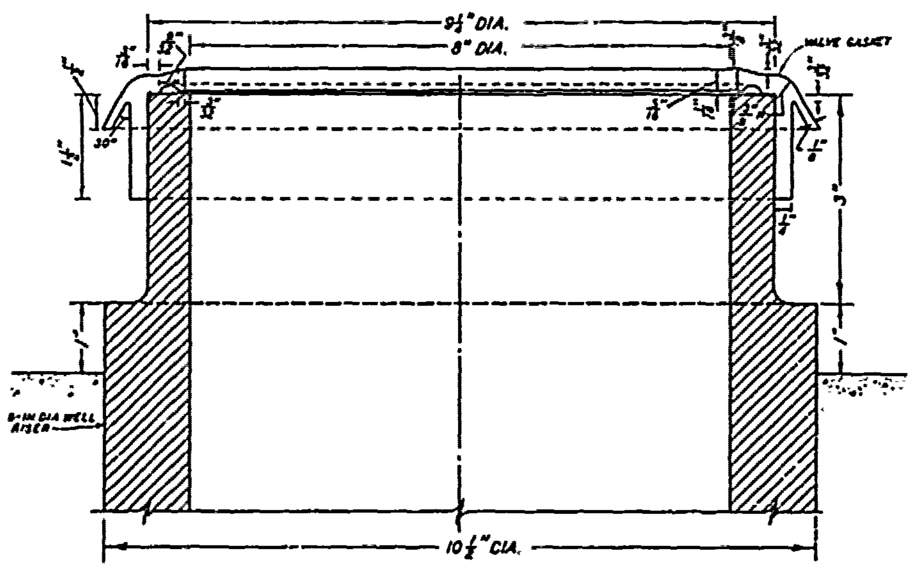
Fig. 90. Relief well with metal well guard



a. Check valve
and gasket



b. Details of check valve

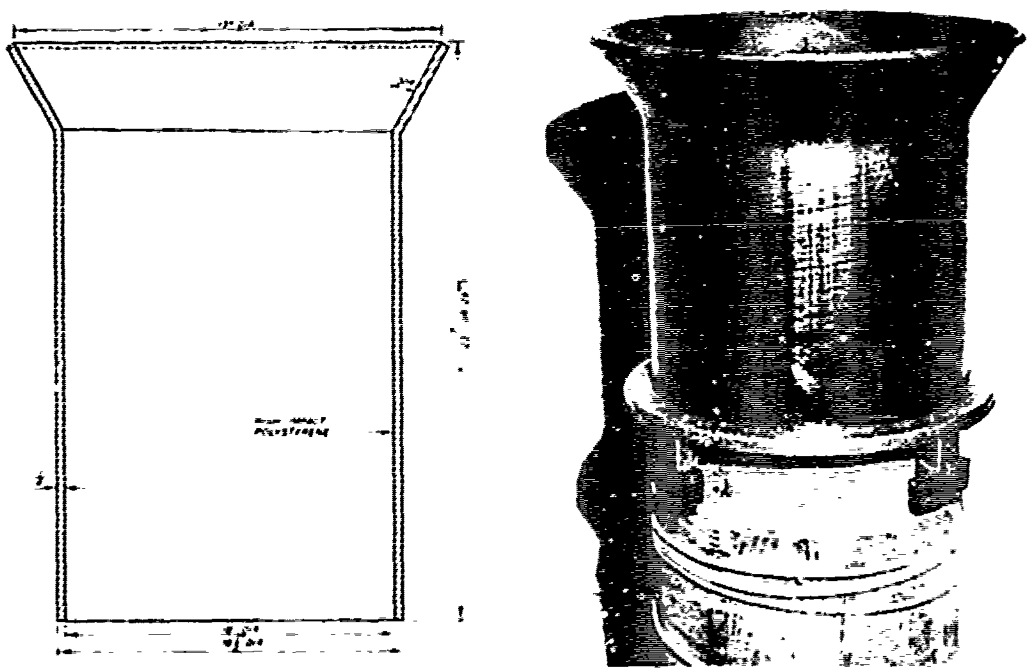


c. Details of rubber gasket

Fig. 91. Details of aluminum check valve and rubber gasket

designed to prevent backflow of surface water into the well and also to serve as a gasket between the well top and plastic standpipe. The mold for this gasket to fit an 8-in. well is owned by the St. Louis District, CE. The gasket should be made of neoprene rubber with a softness of 30 shore. In order for the lightweight check valve to function properly, the top of the well must be made absolutely smooth before the rubber gasket is cemented to it. The gasket can best be attached to the wood or cast iron tenon with an adhesive cement such as 3-M- No. EC-870, Minnesota Mining and Manufacturing Company.

782. Flow from relief wells will be somewhat more than that due to natural seepage during periods of relatively low stages on the levees. In agricultural areas, each well should be provided with a plastic standpipe to raise its discharge elevation 1 or 2 ft above natural ground surface (fig. 92), unless the well will be submerged by surface water or unless flow from the well will drain into natural sloughs or drainageways. (The flare on the standpipe shown in this figure is designed to prevent



a. Details of standpipe b. Standpipe in place on cast-iron tenon

Fig. 92. Plastic standpipe for relief wells

the check valve from catching on the top of the standpipe in the event the well flows before the standpipe is removed.) These low sleeves or standpipes will prevent well flow during periods of low river heads on the levee when no pressure relief is necessary. As soon as artesian pressure develops to such extent that water begins to spill over the tops of the standpipes, they should be removed and the system allowed to operate as originally designed.

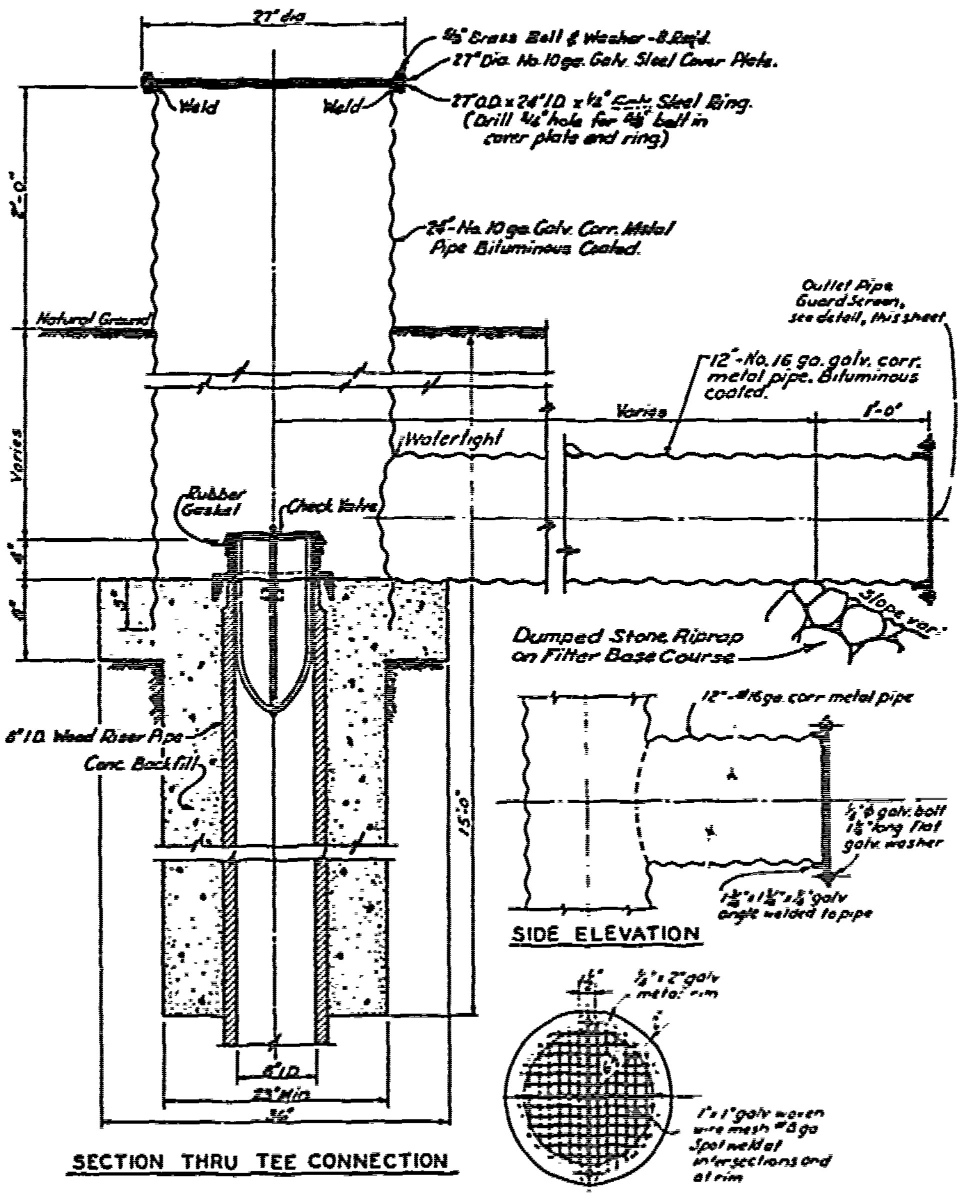


Fig. 93. Tee-type well outlet

783. Relief wells adjacent to ditches or intakes to drainage structures or pumping stations along the levee should be provided with a check valve, standpipe for access, and a horizontal discharge pipe such as shown in figs. 93 and 94.

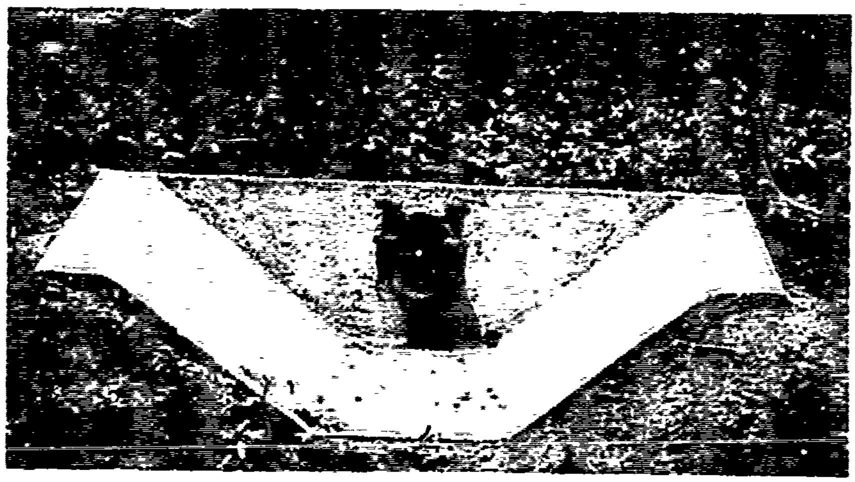


Fig. 94. Tee-type outlet with flap gate and concrete discharge apron

784. Relief wells installed in areas subject to flooding by sanitary or industrial sewage should be provided with the type of rubber gasket and check valve shown in fig. 91, an outfall pipe equipped with an automatic flap gate, and a well top of the type shown in fig. 95.

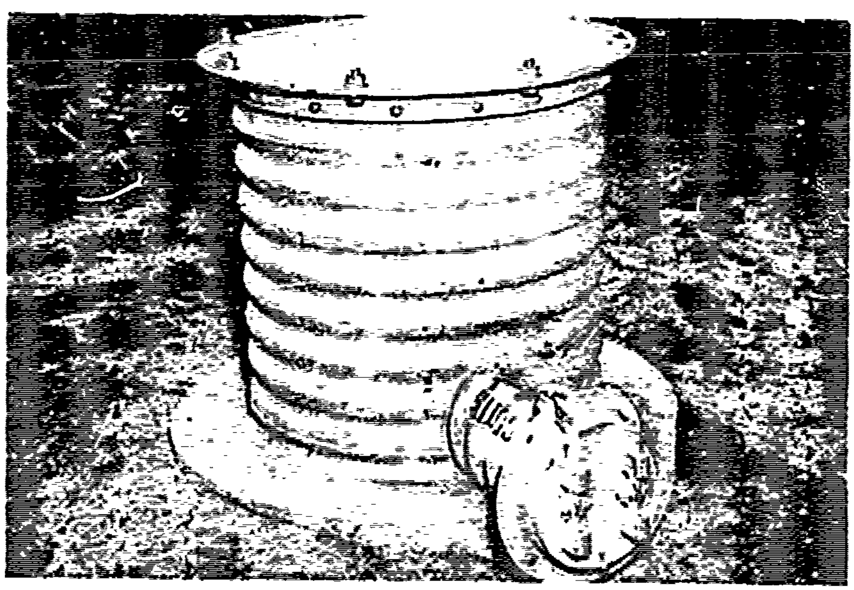


Fig. 95. Well guard and outlet used in areas subject to flooding by sanitary or industrial sewage

These flap gates should be of cast iron with bronze facing. They should be double-hinged and bronze-mounted, and should close with a slight vertical angle. Bolts and washers should also be of bronze or brass.

785. A completed relief well installation in an agricultural levee district in the St. Louis District, CE, is shown in fig. 96.

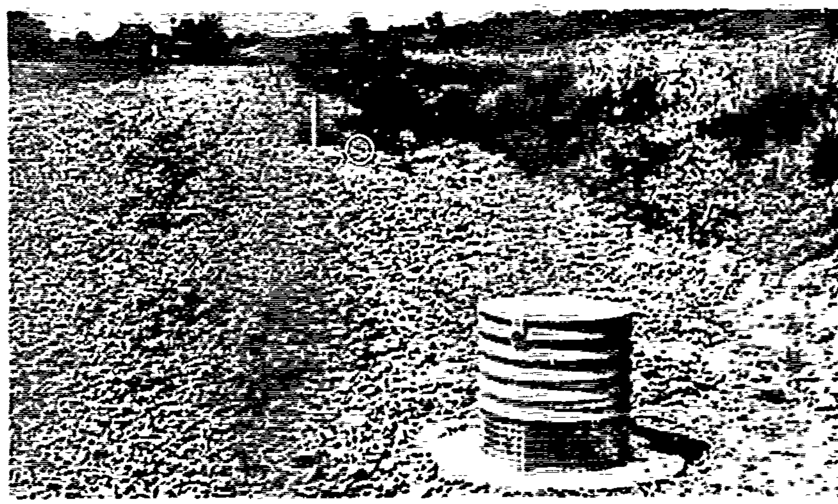


Fig. 96. Line of relief wells with surface-type outlets

Piezometers

Description

786. Piezometers should consist of all-brass or bronze screen set in filter sand, a riser pipe generally extending 3 ft above ground surface, and a cylinder of concrete around the riser pipe extending from just below the piezometer cap to 30 in. below ground surface. The piezometer screen normally will be 24 in. long with a 40-mesh screen or No. 18 or 25 slot size. The riser will generally be commercial grade, standard weight, 1-1/4-in.-ID galvanized steel pipe with couplings of galvanized steel, except that a plastic coupling should be used between the brass screen and riser pipe. The riser pipe should be fitted with a standard-weight galvanized steel cap, with a 1/8- or 1/4-in. hole in its top, attached to the concrete cylinder around the riser pipe (see fig. 97). All joints should be treated with a suitable joint compound

which will seal the joint and completely cover and protect that part of the riser pipe from which the galvanizing has been removed by threading. All joints should be made tight but care should be taken not to damage the plastic coupling while tightening the assembly. The number of the piezometer should be stamped on both cap and riser pipe.

<u>Installation</u>

787. <u>Boring methods.</u> Piezometers should be installed in 6-in. minimum diameter holes drilled to the required depth. The holes must remain open, or be held open with casing if necessary, to allow placement of the sand filter around the piezometer screen and to allow proper backfilling around the riser pipe. The boring for a piezometer may be advanced with an auger or by a bailer and casing.

788. <u>Sampling.</u> Samples should be taken at every change in type of soil and at intervals not to exceed 3 ft, in order to obtain representative samples of the various strata encountered.

789. <u>Filter.</u> The filter material should consist of clean, well-graded medium to fine sand which should extend 6 in. below the bottom of the screen, completely surround it, and extend at least 6 in. above the top of the screen.

790. <u>Backfill.</u> Space around the riser pipe should be backfilled with

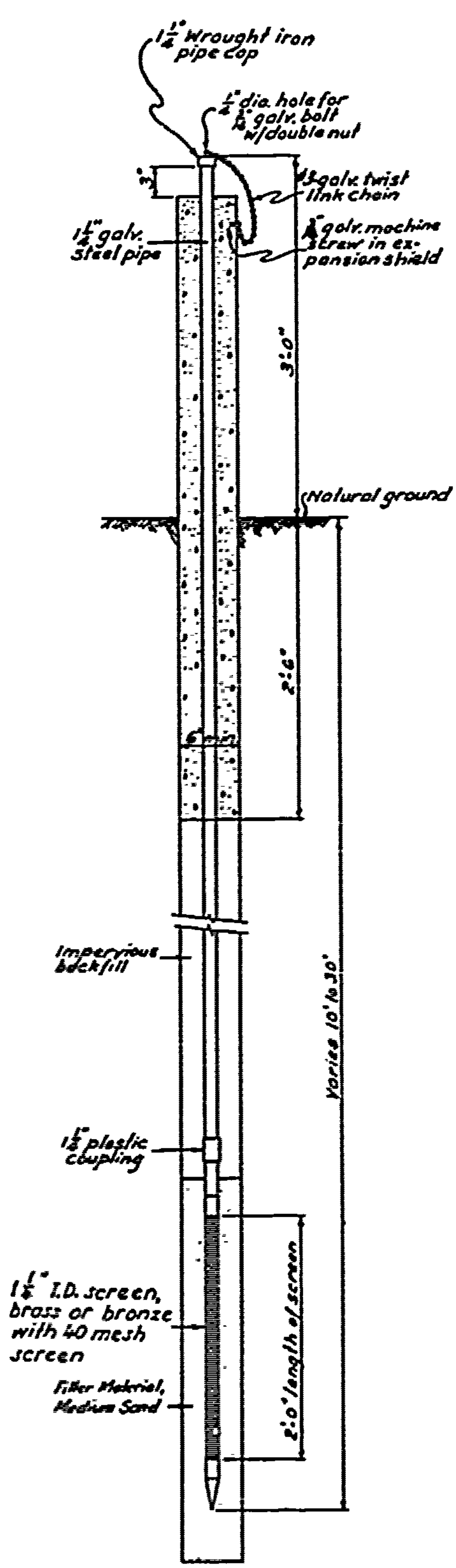


Fig. 97. Piezometer details

impervious soil or concrete from the top of the filter material, or above where sand caves in when casing (if used) is withdrawn, to within 30 in. of the ground surface. Soil backfill should be placed in 4-in. layers and thoroughly compacted. If the space around the riser pipe contains water, it should be filled with concrete placed by the tremie method.

Fig. 98. Piezometer with riser pipe protected by concrete collar

791. A 6-in.-diameter concrete cylinder should be installed around the riser pipe from the top of the compacted backfill to within 3 in. of the top of the piezometer pipe. This cylinder may be formed by placing a 6-in.-round cardboard form from 3 in. below top of riser pipe to 6 in. below ground surface and filling with concrete. A completed piezometer is shown in fig. 98.

792. Testing. After installation is completed, each piezometer should be pumped until a clear stream of water is obtained. If the piezometer screen is above the water table or the water table is too deep to permit pumping, the piezometer should be filled to the top with clear water and the rate at which the water falls in the riser pipe recorded. Depth to water should be recorded every minute for the first 5 min and at 5-min intervals thereafter for the following periods, depending on the soil in which the screen is set.

Piezometer Screen Set In	Period of Observation (min)	Approximate Time for 50% Fall (min)
Sandy silt	30	30
Silty sand	10	5
Fine sand	5	1

If the piezometer has been installed properly, the time required for the water in the riser pipe to fall 50% of the total fall to the ground-water table should not be appreciably, if any, greater than that listed above.

An alternate method of test is to install a 5- to 10-gal graduated container of constant cross section on top of the piezometer, fill the piezometer and container with water and observe the rate of fall of water in the container. Piezometers found to be functioning improperly because of clogging or obstructions should be removed and reinstalled approximately 5 ft distant from the first installation.

793. The approximate elevations of the tips of piezometers usually are furnished to the inspector with the type of soil in which the screen is to be set and/or a soil profile on which the piezometer screen has been plotted to show the stratum in which it is intended the screen should be installed. In general, the screens should be installed in clean sand; in no case should any part of the screen be installed in clay or silt. All installation data should be recorded on a form such as shown in fig. 99.

Seepage Berms

794. Seepage berms can be constructed by hydraulic fill methods or by hauling. The method of construction and type of fill to be used will depend on whether the berm has been designed as an impervious, semipervious, pervious or sand, or a free-draining berm in accordance with the following tabulation.

Type of Berm	Type Soil	Berm Coefficient of Permeability $(k_v) \times 10^{-4}$ cm/sec	Method of Placement	Lift Thickness in.
Impervious	Random	<1	Hauling	12 to 18
Semipervious	Silty sand or sand	>1 <100	Hauling or dredging	--
Pervious	Sand	>100	Hauling or dredging	--
Free-draining	Random and gravel filter blanket	*	Hauling	--

* Seepage-carrying capacity of gravel blanket equal to or more than 1 to 2 gpm per linear foot of berm with a head loss of less than 1 or 2 ft for flow through full width of berm.

PIEZOMETER INSTALLATION REPORT

PROJECT: St. Louis District, Underseepage | LEVEE DISTRICT: Prairie du Rocher

LOCATION (STA.): 125 + 77 | OFFSET FROM CENTER LINE: 128 - ft landside | PIEZ NO.: P-5

PIEZ TYPE: Open 1¼ x 24-in. all brass well point with No. 18 slots | DEPTH OF PIEZ: 10.7- ft | RISER PIPE DIAM: 1¼ in. I.D.

PIEZ TIP SET IN (SOIL TYPE): Brown fine sand | SOIL SAMPLE NO.: 3 | BORING DIAM: 6 - in.

METHOD OF INSTALLATION: Hand auger and casing

TYPE OF PROTECTION FOR PIEZ: 6-in concrete cylinder around riser pipe | VENT: ⅛-in. hole in cap

GROUND ELEV: 381.9 | ELEV TOP OF RISER: 384.78 | ELEV PIEZ TIP: 371.8

FILTER: Medium sand | FROM ELEV: 370.7 | TO ELEV: 374.6

SEAL: Lean clay / concrete | FROM ELEV: 374.6 / 375.6 | TO ELEV: 375.6 / 384.5

INSTALLED BY: Luhr Bros., Inc. | CONTRACT NO.: 4846 | FOREMAN: Reichman

DATE OF INSTALLATION: 7 August 1953 | DATE OF OBSERVATIONS: 14 August 1953

METHOD OF TESTING PIEZ: Falling head

TIME AM	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET
10:22	0	0.00						
10:22¼	¼	12.40						
10:23¼	1 ¼	12.70						
10:24¼	2 ¼	12.90						
10:30	8	12.30*						

REMARKS: * Static water table

Piezometer number stenciled on riser pipe and cap

Wayne A. Smith
INSPECTOR

Fig. 99. Form for recording piezometer installation data

Berms do not require any special compaction other than that resulting from placement and spreading operations. Special precautions must be taken in the construction of free-draining berms to insure that the filter layers are properly constructed and the gravel filter has the required permeability.

Riverside Blankets

795. Riverside impervious blankets should be constructed of the most impervious material readily available. It should be placed in 6- to 12-in. lifts and compacted with controlled movement of the hauling equipment or at least three coverages of a crawler-type tractor exerting a tread pressure of at least 6 psi. The thickness of the blanket will depend upon its permeability and other design considerations but should be a minimum of 3 to 5 ft.

796. Borrow for seepage berms, or for impervious riverside blankets or sublevees as discussed subsequently, should either be obtained at a distance of 1000 to 1500 ft or more from the riverside levee toe, or borrow operations should be controlled so as to leave a blanket of clay or silt at least 5 ft thick over the underlying pervious sands.

797. If the blanket will be subject to very severe scouring action, it should be protected by means of spur and/or abatis dikes strategically located. Borrow pits in which impervious blankets are placed should be drained to permit the growth of willows.

798. Borrow pits that have been excavated to sand for levee construction can be partially refilled with silt by construction of spur and abatis dikes at strategic locations along the levee. Although such a blanket of silt and sand will not be as impervious as a compacted clay blanket, it will materially reduce underseepage and hydrostatic pressures landward of a levee. (Construction of abatis dikes is subsequently discussed.) Collection of silt by means of abatis dikes requires upstream inlet ditches to bring water into the borrow pits at below bankfull stages and downstream outlet ditches to permit drainage of the pits after the high water has receded. Drainage of the pits is required to permit

the growth of willows which are necessary if the pits are to be filled by siltation. The growth of willows is necessary to replace the abatis dikes which have an expected life of no more than 3 to 5 years.

Sublevees

799. Sublevees constructed around critical seepage areas should have a minimum crown width of 5 ft and side slopes of 1 on 2-1/2 or 3. They should be built of relatively impervious soil compacted in 9-in. lifts with at least three coverages of a crawler-type tractor or the equivalent by controlled movement of hauling equipment. Borrow for sublevees should never be obtained between the levee and the sublevee. The material should be placed within 2 to 4% of optimum moisture to insure its stability when subjected to a head of water and to minimize the development of shrinkage cracks. The base for the sublevee should be properly prepared so as to prevent development of piping beneath the sublevee. Sublevee basins should be provided with paved overflow spillways to prevent crevassing in event of overflow as a result of seepage or surface runoff from the main levee. A gated outlet should also be provided to drain the basin after a high-water period. The sublevee basin should have at least 1 ft of freeboard above the maximum anticipated pool elevation with the spillway in operation. Sublevee basins longer than about 300 to 500 ft should be provided with cross dikes and each basin thus formed should be provided with an overflow spillway. The sublevees and dikes should be sodded and maintained.

Abatis Dikes

800. Abatis dikes usually consist of a row of posts across the borrow pit that support 2- by 4-in. stringers. To these stringers are stapled field fencing and snow fence as shown in fig. 100. The dike is braced with 2- by 4-in. bracing anchored to stakes driven into the ground on 18-ft centers. In order to prevent undercutting of the dike as result of scour, it should be protected both upstream and downstream by means of a riprap mattress.

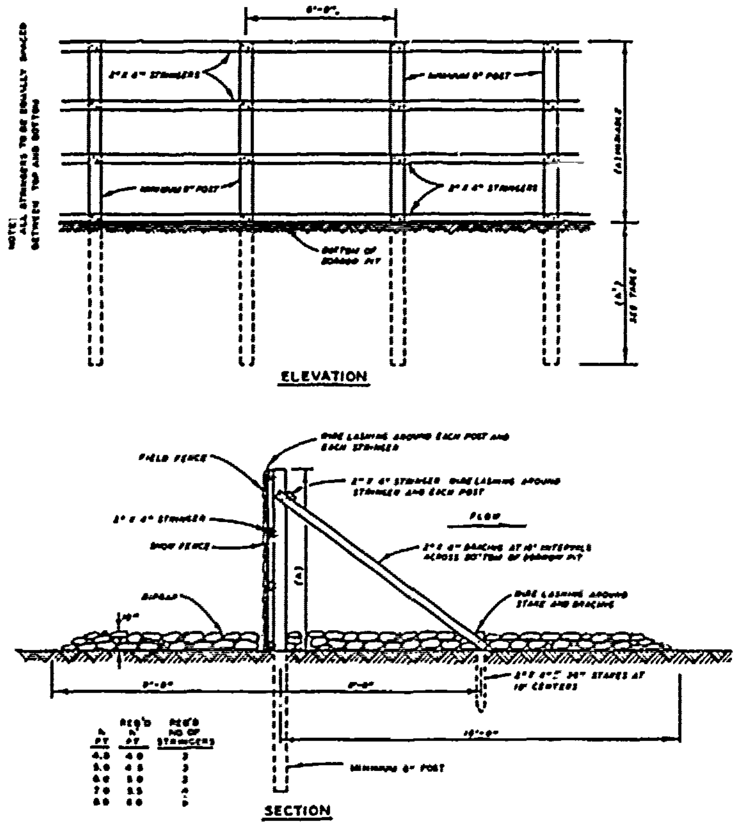


Fig. 100. Details of abatis dike

801. Generally, abatis dikes vary in height from 4 to 8 ft. The length and depth of supporting posts are usually varied in proportion to the height of the dike as shown in fig. 100. Prior to construction of a dike, minor grading and leveling is usually required in the bottom of the borrow pit along the dike line. The 2- by 4-in. stringers and braces may be of either hard or soft wood, preferably No. 2 common grade or better. Willow or cottonwood lumber is not acceptable. The posts should be 6 in., either round or square, and should be sound and free of defects that would impair their effectiveness for the purpose intended. Willow or cottonwood posts are not acceptable. The snow fence usually consists of 1-1/2- by 12-in. slats, with a spacing of about 2 in. between laths. Laths should be painted with one coat of red oxide, or equivalent, paint. The field fencing should have top and bottom wires of not less than No. 9

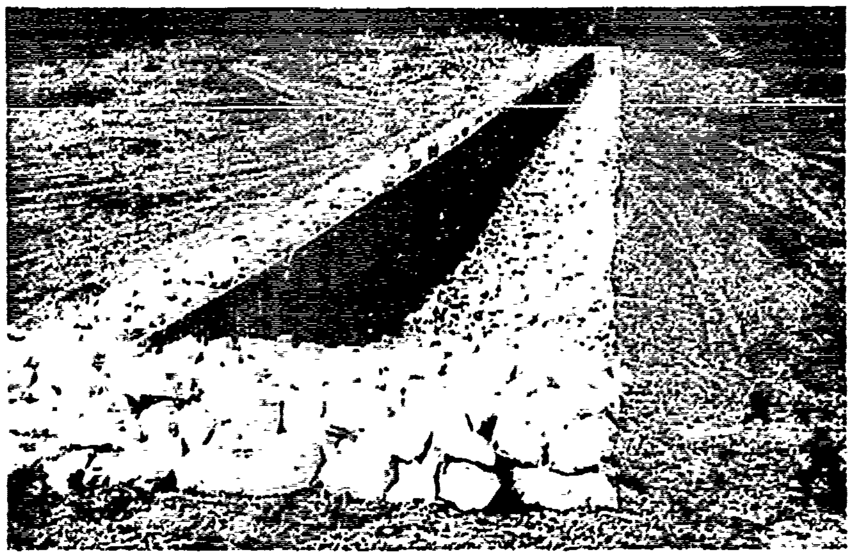


Fig. 101. Abatis dike across borrow pit, Columbia Levee District.
St. Louis District, CE

gage wire, and bars and staves should be not less than No. 11 gage wire. The staves and bars should be spaced on about 6-in. centers. The width of the fencing should be not less than the width between the adjacent dike stringers. The fencing should also be zinc-coated. The riprap stone should be of durable quality and should grade from about 3 to 150 lb.

802. Fig. 101 shows a completed abatis dike.

Drainage Blankets

803. As proper functioning of a drainage blanket depends upon its continued perviousness, they should not be constructed until after the landside slope of the levee has become stabilized and covered with sod so that soil particles carried by surface runoff and erosion from the levee will not clog the blanket. If it is necessary to construct the blanket at the time the levee is built or before it has become covered with sod, an interceptor dike should be built at the intersection of the landside levee slope and drainage blanket to prevent surface wash from clogging the blanket. The flow intercepted by the dike should be drained off through spaced outfall channels. After the levee has become covered with sod, the interceptor dikes and outfall channels should be removed and the blanket completed.

804. A drainage blanket should not be constructed unless the pervious foundation extends completely to the surface. If sod or vegetation exists in the area to be covered by the blanket, they should be removed before the blanket is placed. The loss of material involved may be made up with locally available clean sand, or the first filter layer may be increased in thickness.

Drainage Trenches

805. Drainage trenches are excavated along the landside toe of a levee for the purpose of tapping the underlying pervious substratum. In order to be effective, the trench must penetrate any upper strata of clay, silt, or very fine sand. Although the depth of the trench will usually be set on the basis of previous borings, soil strata frequently vary in relatively short distances and therefore it may be necessary to adjust the depth of the trench during construction in order to insure that it taps the main underlying aquifer.

806. Drainage trenches are usually excavated immediately landward of the levee toe. Side slopes of such trenches should be as steep as possible (usually 1 on 1 or 1 on 1.5) but must be stable so as not to endanger the levee slope. The trenches should be constructed in the summer or fall when the water table is lowest; even then it may be necessary to use a dewatering system to achieve the required depth for the trench.

807. For a drainage trench to function properly, it is imperative that the filter layers be properly graded and be carefully placed. Likewise, the collector pipe should be properly installed. There should be no joints with openings larger than the perforations in the pipe. Precautions should be taken to prevent the filter layers or collector pipe from becoming flooded with muddy surface water during construction.

808. The excavation for the trench may be refilled with material taken from the trench and compacted with either a crawler-type tractor or by hauling equipment.

Cutoffs

809. Where the depth of the pervious substratum is less than about 40 ft, an impervious cutoff can be constructed at the riverside toe of the levee by means of either open excavation or a trenching machine and backfilling with impervious soil. The cutoff should be placed either beneath the levee (in the case of new levees) or at the riverside toe. If placed at the riverside toe it should be tied into the levee by means of an impervious blanket.

810. Cutoffs may be constructed of earth, steel sheet piling, or possibly by grouting through grout holes on close centers; however, earth construction is considered the most practical method. Although steel sheet piling is commonly used for cutoffs, experience has indicated that it is not entirely watertight as leakage occurs through interlocks, at splices, and through torn interlocks. Unless the sheet pile cutoff is tight and continuous, little reduction of pressure landward of levees can be expected. However, a sheet pile wall properly driven will significantly reduce the danger of piping beneath the levee. The high cost of steel sheet piling generally precludes its use for deep cutoffs. Grouting of alluvial sands has been attempted but is quite expensive and at its present stage of development cannot be considered as a practical means of cutting off a deep pervious aquifer.

811. An open excavation for a cutoff should be cut on slopes which will insure stability of the slopes and the levee. A wellpoint system will usually be required where the excavation extends below the water table. The excavation should be backfilled with impervious soil (clay or silt) compacted in layers by means of a crawler-type tractor or controlled movement of hauling equipment.

812. A cutoff can also be constructed by dragline or specially built trenching machine capable of digging trenches 40 or 50 ft deep. In this method of construction the trench is held open by keeping it filled with a clay slurry. It is essential that the clay content of the slurry be sufficient to coat and seal the walls of the trench in order that the hydrostatic pressure can be effective in holding the trench

open. The clay slurry can be prepared by chopping the clay with a disk harrow and mixing it with water by means of pumps and/or paddles. The trench is then filled with the slurry. In order that the procedure be successful, the operations of digging, keeping the trench filled with slurry, and backfilling must all be carried out simultaneously.

813. Where a levee is underlain by a relatively thin deposit of natural levee or crevasse sand, in turn underlain by impervious strata, the sand stratum can be cut off by either of the above procedures. Construction of such a cutoff along a levee in the St. Louis District is illustrated in fig. 102.

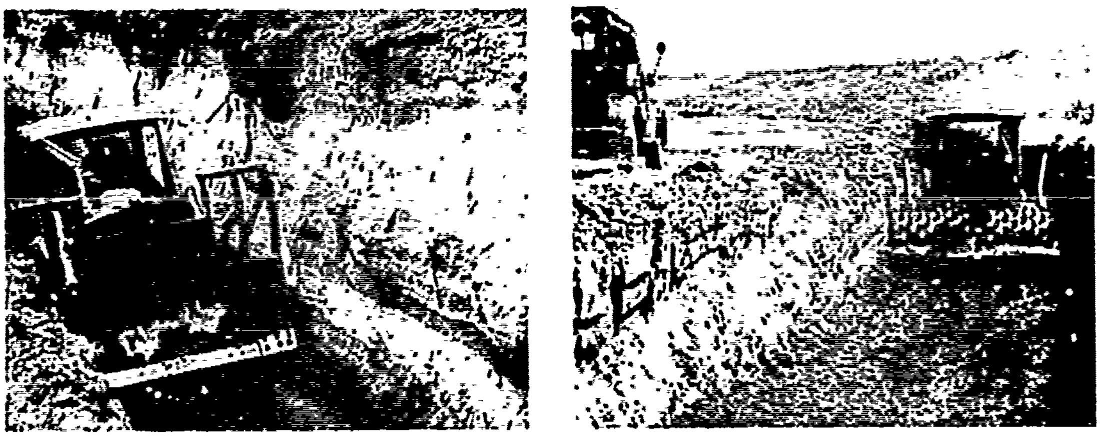


Fig. 102. Construction of cutoff through shallow sand stratum beneath levee

PART VIII: MAINTENANCE OF UNDERSEEPAGE CONTROL MEASURES AND OBSERVATIONS DURING HIGH WATER

Maintenance

Relief wells

814. Relief wells require a certain amount of nominal maintenance to insure their continued and proper functioning. In order for relief wells to function properly, it is essential that they be kept free of sand, silt, organic matter, or any other material that would retard free flow. Relief wells should be inspected once a year, preferably immediately prior to normal high-water seasons, and more often during major high waters for the purpose of detecting vandalism, theft, abuse by thoughtlessness or carelessness, unauthorized use of the wells or piezometers, or other irregularities. Each well should be sounded every two or three years and after each major high water to see that it is free of trash and any obstruction, and to determine the amount of sand, silt, or other material that may have settled at the bottom of the well. Any trash, obstruction, or sediment to a depth greater than 1.5 ft in the well should be removed.

815. All wells requiring removal of sediment should be pump-tested after cleaning to see if any appreciable loss of efficiency has resulted from foreign material entering the well. In addition, all wells should be pump-tested at least every 5 to 8 years. If the pumping test indicates that the specific yield is 20% or more lower than the original specific yield, the well should be surged in an effort to restore its original efficiency. Individual wells known to have been subjected to inflow of muddy water as a result of inoperative check valves or removal of check valves or vandalism should be pumped and/or cleaned before the next high-water season.

816. Silt, sand, or organic material that may have accumulated around the top of a relief well should be removed. Check valves and flap gates should be maintained so that they will operate properly and close securely. Sand or other material that may have accumulated in or

around flap gates so as to obstruct the flow or prevent functioning of the gates should be removed. Outfall ditches, bank slopes, or berms should be properly maintained in the vicinity of horizontal outfall pipes.

817. During each inspection special attention should be given to check valves, gaskets, and standpipes, and each component should be maintained in the same working condition as when it was initially installed.

818. The area in the immediate vicinity of the wells should be kept free from weeds, trash, and debris. Mowing and weed spraying should be extended at least 5 ft beyond the well and the ground shaped and maintained for inspection and servicing of the wells.

819. Relief wells may be used for water supply purposes, provided they are not overpumped and not damaged in any way as a result of pumping operations. Any connections to relief wells for water supply purposes should be of a flexible nature that will permit normal operation of the well during high-water periods.

820. A record should be kept of all inspections and maintenance performed.

Seepage berms

821. Seepage berms require relatively little maintenance. They should be kept graded to drain, and any large gullies that develop should be filled. Objectionable growth of weeds or brush should be controlled by grazing or periodic mowing.

Riverside blankets

822. Riverside blankets should be inspected after high water to check on scour. If the blanket has been damaged by scour, the scour should be repaired and the blanket protected by means of properly placed abatis dikes and by encouraging the growth of willows.

Sublevees

823. Sublevees should be inspected periodically to check their grade and section, and the condition of the control gates, spillways, and drains. Sublevees and the area within them should be maintained free of objectionable weeds and brush, any erosion of the sublevees should be repaired, and control gates, spillways, and drains should be kept in operating condition at all times.

Cutoffs

824. Cutoffs require no inspection or maintenance.

Drainage trenches

825. Drainage trenches of the design illustrated in Part VI should be given the applicable inspection and maintenance described for relief wells.

Piezometers

826. All piezometers should be inspected annually for damage or any unusual condition that might affect their performance. The site of piezometers should be kept clear of weeds and brush and cared for in the same manner as described for relief wells. Any damage to or maintenance performed on piezometers should be reported, and the piezometers repaired.

Underseepage Measurements and Observations during High Water

Piezometer and seepage observations

827. When river stages higher than 8 ft on the levee are predicted, plans should be made for reading piezometers at least twice a week until after the crest of the flood has passed and the river stage fallen below 8 ft.

828. Seepage observations should also be made when the piezometers are read for the purpose of reporting the location and severity of sand boils and other underseepage conditions in the area. Seepage should be classified as light, medium, or heavy, and the area in which it is emerging recorded as well as the head on the levee at the time. The location of sand boils larger than 4 in. should be noted by recording the levee stationing and distance from the center line of the levee or landside toe of levee or berm.

829. If seepage water or surface water covers the area adjacent to piezometers or where seepage or sand boils are occurring, the elevation of such water should be recorded.

Operation and obser-
vation of relief wells

830. Normal well operation requires removal of standpipe as soon as the well overflows the standpipe. Check valves and gaskets are left on the wells at all times. Flow from selected wells and reaches of wells should be measured periodically as a check on design computations and to provide a record of the amount of water being discharged. The flow from the wells can best be measured by means of the relief well meter described in reference 52, or it may be approximated roughly by observing the height of water flowing from the well above the top rim of the riser pipe.

Sublevee basin observations

831. The elevation of water in sublevee basins should be determined at sites where piezometers are installed whenever the piezom are read. During major high water, the elevation of water i basins should be recorded. Seepage flow over spillways in sublevee basins should be estimated during high water.

Seepage flow measurements

832. The natural seepage flow should be measured at the Gammon, Commerce, Trotters 51 and 54, Stovall, and Baton Rouge piezometer sites, at the same points where it was measured in 1950, at river stages of 10, 15, 20, and 25 ft on the levees. Flow from any new relief wells, sublevee basins, or landward of new seepage berms (where practicable), at the piezometer sites should also be measured at these river stages and at the first 5-ft stage.

PART IX: EXAMPLES OF DESIGNS FOR SEEPAGE CONTROL MEASURES AT PIEZOMETER SITES STUDIED

833. Examples of designs for seepage control measures have been prepared for those piezometer sites where additional measures are considered necessary based on analyses presented in Part . The designs are applicable only to the particular reach of levee covered by the investigation. The method or, in some cases, alternate methods of control for which designs have been prepared are those considered most applicable to that particular site from an engineering and construction standpoint. The primary purpose in presenting these designs is to illustrate the factors that enter into the selection of a control measure and its design, and to compare the design of different control measures. The bases of design and the designs of recommended control measures are given in tabular form for each site where control measures are considered necessary. Selection of the control measures to be used should largely be based on cost and availability of right-of-way. Right-of-way for riverside blankets can usually be readily obtained. Relief wells require about 10 ft of right-of-way along the landside toe of the levee with a 50-ft easement during construction. Seepage berms, because of their width, require considerably more right-of-way.

834. The designs presented are based on geological and soils data, topographical features, piezometric and permeability data, and formulas presented in the preceding parts of this report. All designs are based on a project flood stage. The riverside blankets, relief wells, and berms recommended will probably not prevent sand boils from occurring in all instances but are believed adequate to insure the safety of the levee.

835. Relief well system designs are based on the use of 8-in. ID wooden wells with a 6-in. gravel filter, and an effective penetration of 50% of the pervious aquifer (60% on a depth basis). The required length of well screen $\overline{W}$ shown in the tables is the total length required for 50% effective penetration of the sand aquifer. At some sites, fine sands exist above the top of the main screen, and "extra" screen to penetrate

and intercept seepage in the fine sand strata is indicated in the tables. (If during the installation of relief wells it is found from samples of the drilling effluent that the sands are too fine, the "extra" screen should not be installed.)

836. At several sites where relief wells are recommended, only limited data pertaining to the thickness and gradation of the foundation sands are available. In general, the thickness of the pervious substratum should be obtained about every 1000 to 1500 ft by means of seismic methods or deep borings. Borings with a depth approximately equal to the depth of the wells should be made on about 1000-ft centers in reaches where wells are to be installed, to determine the gradation of the sand strata in which well screens would be set and the required length and depth of well screen.

837. The required spacings of relief wells were determined from figs. 63 through 66, which are based on $k_f = 1250 \times 10^{-4}$ cm per sec and $D = 100$ ft. Where the well reach is fairly short, well spacings were reduced using fig. 62. The estimated well flows were obtained from these figures but were adjusted to correspond to the values of k_f and D at the site.

838. The tops of relief wells would be set about 4 in. above the natural ground surface, and provided with a rubber gasket, check valve, well guard, and plastic standpipes as described in Parts VI and VII. The height of standpipe above the natural ground surface may equal 0.25 z_t but should not exceed 3 ft. Collector ditches for the wells are not recommended at any of the sites discussed because they reduce the thickness of the top stratum and thereby decrease the required well spacing. Where the locations of relief wells are included in the tables, the stationing refers to that along the center line of the levee with the wells offset perpendicular to the center line. The wells generally would be located about 5 ft landward from the toe of the existing levee or berm.

839. The designs of riverside blankets are based on formulas given in Part VI. The riverside conditions at the site, assumed in the design, are identified by a case number corresponding to the case numbers used in Part VI.

840. Landside seepage berms were designed using the formulas in fig. 67, and in accordance with paragraph 727, page 314. In several instances the computed width of a berm required to reduce the substratum pressure at the berm toe to an allowable amount was considerably in excess of 400 ft. However, the maximum width of berm recommended is 400 ft. As the recommended width of some berms is less than the computed required width, certain of the berms will not give pressure relief comparable to that provided by relief wells or riverside blankets, as the latter designs were always based on reducing the upward gradient through the top stratum landward of the levee to 0.5 or 0.6.

Caruthersville, Missouri

841. The maximum H at the Caruthersville site is relatively low and, as the landside top stratum is relatively uniform, the seepage problem is not considered particularly serious. Consequently it is believed that some additional blanket in the rather open riverside borrow pits will be adequate for the control of seepage at the site. The design of the required blanket is given in table 41.

842. Formulas are not available for design of riverside blankets for conditions existing at Caruthersville; however, the conditions fall into a category close to case III and this case was used to compute the required riverside blanket. On the basis of the analyses shown in table 41 it appears that a silty blanket 3 to 5 ft thick with a permeability of 1.5×10^{-4} cm per sec would provide adequate seepage control between sta 25/40 and 26/32. This blanket could probably be developed in time by siltation between abatis dikes and in willow growth in the borrow pits. It is recommended that abatis dikes be constructed at 500-ft intervals between sta 25/40 and 26/32 with their tops at el 265.0 An inlet channel would have to be excavated from the river to the borrow pit upstream of sta 25/40, and an outlet channel from the pit to the river downstream from sta 26/32, to bring water and sediment into the borrow pits at relatively low river stages. The invert of the inlet channel should be at el 258; that of the outlet channel at el 262. The riverside edge of

Table 41

Summary of Designs of Seepage Control Measures

Caruthersville, Mo., Site

Item	Stationing of Reach 25/40-26/9	Stationing of Reach 26/9-26/16	Stationing of Reach 26/16-26/32
Design Factors			
h_o in ft	6.1	4.2	5.1
H in ft	19.2	16.7	16.7
s in ft	500	500	500
x_3 in ft	230	170	220
z_t in ft	7.0	4.5	8.0
d in ft	100	100	100
k_f in 10^{-4} cm/sec	1500	1500	1500
Avg ground el	268.0	270.5	270.5
L_3 in ft	∞	∞	∞
Q_s in gpm/100 ft	625	560	----
Riverside Blanket			
Case	III	III	III
Type	Silt	Silt	Silt
i_a	0.5	0.5	0.5
h_a in ft	3.5	2.3	4.0
x_r in ft	700	790	400
L_B in ft	300	300	300
z_{bR} in ft	5	0	0
k_{bR} in 10^{-4} cm/sec	26	----	----
k_B x 10^{-4} cm/sec	1.5	1.5	1.5
Computed z_B in ft	4.6	6.2	1.6
Recommended z_B in ft	5	6	3
Q_s in gpm/100 ft	335	290	400

the traverse in the borrow pit at about sta 26/20 should be degraded to el 262 to permit river water to enter the pit downstream from the traverse at low stages.

843. A more effective blanket could be obtained by placing a clay blanket in the borrow pits with sufficient thickness to result in a combined thickness of blanket and existing clay top stratum of 3 to 5 ft. As would be done for the silt blanket, the borrow pits should be drained to promote the growth of willows to protect the blanket against scour.

Gammon, Arkansas

844. For purposes of comparison, a riverside blanket, a system of relief wells, and extensions to the present landside seepage berm have been designed for various reaches of the Gammon levee as shown in tables 42-a and 42-b. Design factors and control measures shown in table 42-a are based on the assumptions the landside drainage ditch would not be filled; those in table 42-b are based on the assumption the ditch would be filled.

845. In the design of the riverside blankets it was assumed that the formulas for case IV were applicable to riverward conditions at Gammon, as the levee is about 2-1/2 miles from the river and the top stratum riverward of the borrow pits consists of relatively thick, impervious clays. The computed thickness of a compacted clay blanket necessary to reduce substratum pressures beneath the landside drainage ditch to tolerable amounts was less than 1 ft. Thus a minimum thickness (3 ft) of blanket is considered adequate. It is estimated that about 4000 to 5500 cu yd of blanket material would be required per 100 ft of levee. Clay for such a blanket could be obtained riverward of the existing borrow pits, but operations necessarily would have to be controlled to insure that at least a 5-ft thickness of clay is left riverward of the present borrow pit. The recommended blanket is considered adequate for seepage control without filling the landside drainage ditch; however, filling of the ditch would be desirable.

846. The required spacing of a line of relief wells installed along the toe of the present seepage berm is given in tables 42-a and 42-b. For purposes of comparison, the spacings in table 42-a were based on the assumption that the landside ditch would not be filled and those

Table 42-a

Summary of Designs of Seepage Control Measures

Values Assuming Landside Drainage Ditch not Backfilled

[illegible], Ark., Site

Item	Stationing of Reach					
	138/4-138/15	138/15-138/45	138/45-139/8	139/8-139/13	139/13-139/18	139/18-139/2[illegible]
Design Factors						
h_0 in ft	10.7	11.1	10.2	11.6	13.1	10.2
H in ft	25.2	26.2	26.2	26.2	26.2	26.2
s in ft	1000	600	1000	1000	2000	1000
x_3 in ft	740	440	640	600	550	640
z_t in ft	5.0	4.0	6.0	8.0	10.0	6.0
d in ft	130	135	135	135	135	135
k_f in 10^{-4} cm/sec	1000	1000	1000	1000	1000	1000
Avg ground el	219.0	218.0	218.0	218.0	215.0	218.0
L_3 in ft	300	300	500	500	500	500
Q_s in gpm/100 ft	330	-500	----	360	----	----
Riverside Blanket						
Case	IV	IV	IV	IV	IV	IV
Type	Clay	Clay	Clay	Clay	Clay	Clay
i_a (toe of existing berm)	0.6	0.6	0.6	0.6	0.5	0.6
h_a in ft	3.0	2.4	3.6	4.8	6.0	3.6
x_r in ft	4970	3850	3500	3040	2870	3500
L_B in ft	350	450	450	450	450	450
z_{bR} in ft	2	1	3	3	3	3
k_{bR} in 10^{-4} cm/sec	8	100	1.6	1.6	1.6	1.6
k_B x 10^{-4} cm/sec	0.05	0.05	0.05	0.05	0.05	0.05
Computed z_B in ft	0.6	0.6	0.4	0.4	0.3	0.4
Recommended z_B in ft	3.0	3.0	3.0	3.0	3.0	3.0
Q_s in gpm/100 ft	80	100	115	120	135	115
Relief Wells						
i_a (toe of existing berm)	0.6	0.6	0.6	0.6	0.6	0.6
h_a in ft	3.0	2.4	3.6	4.8	6.0	3.6
Computed spacing in ft	120	50	140	200	250	140
Recommended spacing	120	75	140	150	250	140
W in ft	95	90	90	90	90	80
El top of "extra" screen	190	200	200	200	200	----
El top of design screen	175	190	190	190	190	185
Depth of well in ft	125	110	110	110	110	115
Q_w in gpm	420	480	495	545	760	495
Standpipe ht in ft	1.0	1.0	1.5	2.0	2.0	1.5
H_w in ft	3.0	3.4	3.6	4.0	6.0	3.6

in table 42-b were determined assuming that the ditch would be filled (table 42-b).

847. If the landside ditch is not filled, relief wells about 110 ft deep on 75- to 250-ft centers would be required along the toe of the present berm. Between sta 138/15 and 138/45, the computed spacing was only 50 ft. However, it is believed that, if wells are installed on

Table 42-b

Summary of Designs of Seepage Control Measures

Values Assuming Landside Drainage Ditch Backfilled

Gammon, Ark., Site

Item	Stationing of Reach					
	138/4-138/15	138/15-138/45	138/45-139/0	139/0-139/13	139/13-139/18	139/18-139/2[illegible]
Design Factors						
h_o in ft*	13.2	11.9	11.2	12.2	13.1	11.2
H in ft	25.2	26.2	26.2	26.2	26.2	26.2
s in ft**	750	350	750	750	750	750
x_3 in ft	1100	500	750	870	1000	750
z_t in ft	7.0	5.0	7.0	8.0	10.0	7.0
d in ft	130	135	135	135	135	135
k_f in 10^{-4} cm/sec	1000	1000	1000	1000	1000	1000
Avg ground el	219.0	218.0	218.0	218.0	218.0	218.0
L_3 in ft	300	900	2000	-	-	-
Q_s in gpm/100 ft	----	----	----	----	----	----
Relief Wells						
i_a (toe of existing berm)	0.6	0.6	0.6	0.6	0.6	0.6
h_a in ft	4.2	3.0	4.2	4.8	6.0	4.2
Computed spacing in ft	160	75	170	190	250	170
Recommended spacing	160	75	175	175	250	175
$\bar{W}$ in ft	95	90	90	90	90	80
El top of "extra" screen	190	200	200	200	200	----
El top of design screen	175	190	190	190	190	185
Depth of well in ft	125	110	110	110	110	115
Q_w in gpm	550	500	600	610	750	600
Standpipe ht in ft	1.5	1.0	1.5	2.0	2.0	1.5
H_w in ft	4.2	3.0	4.5	4.5	6.0	4.5
Seepage Berm						
Type	S1 S4	S1 S4	S1 S4	S1 S4	S1 S4	S1 S4
i_o (toe of levee)	0.5	0.5	0.5	0.5	0.5	0.5
i_1 (toe of new berm)	0.8	0.8	0.8	0.8	0.8	0.8
h_a in ft (toe of new berm)	5.6	4.0	5.6	6.4	8.8	5.6
Computed X in ft**	1460	990	830	806	530	990
Computed t in ft†	8.6	10.1	6.8	6.6	5.6	8.6
Recommended X in ft**	400	400	400	400	400	400
Recommended approx t in ft†	7.3	10.0	7.3	7.3	7.3	7.3
Distance from center line levee to berm crown	550	550	600	600	600	600
El berm crown	221	220	220	220	220	220
Berm slope	1 on 75	1 on 50	1 on 75	1 on 75	1 on 75	1 on 75
$h'_o - t$ in ft	5.1	5.8	5.3	6.0	7.7	5.3
Q_s in gpm/100 ft	340	600	350	340	300	350

* At toe of existing berm.
** Measured from or at toe of levee proper; for design of relief wells values of s in table 42-a were used.
† Measured above average ground surface elevation listed above.

75-ft centers in this reach they will provide enough pressure relief and seepage interception to safeguard the levee from underseepage pressures.

848. Filling the landside drainage ditch to the natural ground surface (about el 219 to 220) will increase the thickness of the top stratum. However, such filling will increase x_3 to some extent. In

general, the computed and recommended spacings for wells with the ditch backfilled are slightly greater than if the ditch were left in existence. It is estimated that about 600 cu yd of material would be required to fill the ditch per 100 ft of levee. Thus, if it is decided to install relief wells, the feasibility of backfilling the ditch should be based on its cost vs the cost of additional wells with the ditch.

849. Consideration also was given to the possibility of extending the present landside berm, as the existing berm is considered to be too thin and narrow. In calculating the dimensions of a new berm (existing berm plus extension), the distances to the effective source of seepage and computed width of new berm were referred to the toe of the levee proper. The thickness of the berm is that at the toe of the levee proper. In the design of the berms it was assumed that the existing drainage ditch would be filled. The computed widths of the berm range from about 500 to 1500 ft (see table 42-b). However, a berm with a total width of 400 ft is considered adequate to insure the safety of the levee against underseepage provided the landside drainage ditch is filled. The berm extension preferably should be built of semipervious or pervious material. It is estimated that about 4500 cu yd of additional material would be required for the berm and backfill in the ditch per 100 ft of levee. Four relief wells would be required on about 100- to 170-ft centers between sta 138/8 and 138/12 even if the berm were extended, as seepage will concentrate in the area of thin top stratum between the toe of the new berm and the landside clay-filled channel.

850. The most efficient method of seepage control at Gammon is considered to be either a line of relief wells along the toe of the present berm, or a compacted clay blanket in the riverside borrow pits. To widen and thicken the present landside berm and fill the landside ditch would require about the same volume of material as that required in a riverside blanket. However, the haul distance would be shorter for a riverside blanket than for a berm, and the blanket material (clays) appears more prevalent than the sandy materials recommended for the berm. If a riverside blanket is selected as the seepage control measure, the borrow pits should be drained to promote growth of willows.

Commerce, Mississippi

851. Substratum pressures at Commerce can be reduced to safe values by a line of relief wells from sta 22/45 to 23/29. From sta 22/45 to 23/25 the wells should be installed on 150-ft centers; from sta 23/25 to 23/29 they should be spaced somewhat closer, say 100 to 125 ft, because soil conditions will cause seepage to concentrate along this reach of levee. The relief wells would have to be about 120 ft deep at Commerce. Required screen lengths for the wells and estimated well flows are shown in table 43.

852. In lieu of relief wells, an extension could be made to the present 200-ft-wide landside berm. The berm extension would have to be constructed of either sand or clay as semipervious materials are not available. The designs and recommended dimensions of both types of berm extension are given in table 43. The computed width of a sand berm extension is about 410 ft; however, on the basis of reasoning previously expressed, it is believed that a 200-ft extension, which would result in a total berm width of 400 ft, would provide adequate seepage control. The sand berm extension would contain about 2000 cu yd per 100 ft of levee; the sand would have to be dredged from the river. The computed thickness of a berm extension of clay would be more than that of a sand berm because a clay extension would not permit seepage through it with attendant pressure relief. The computed width of a clay berm extension is about 1000 ft assuming no leakage through the berm. However, as a berm extension 200 ft wide would force the point of seepage emergence about 400 ft landward from the toe of the levee proper, a 200-ft extension is considered adequate. Although not considered as efficient, a clay berm extension 200 ft wide could be used instead of a sand berm extension. The clays could be obtained at the riverside edge of the existing riverside borrow pit above el 197 and from the abandoned levee.

853. From sta 23/12 to 23/29, the above-described berm extensions need only overlap by 25 to 50 ft the riverward edge of the clay-filled slough immediately landward of the levee. Thus, the recommended extensions could be shortened somewhat along this reach of levee if borings

Table 43

Summary of Designs of Seepage Control Measures

Commerce, Miss., Site

Item	Stationing of Reach 22/45-23/29		Stationing of Reach 23/25-23/29
Design Factors			
h_o in ft	9.7		12.6
H in ft	22.7		22.7
s in ft	800		800
x_3 in ft	600		1000
z_t in ft	7.0		7.0
d in ft	165		165
k_f in 10^{-4} cm/sec	1000		1000
Avg ground	197.5		197.5
L_3 in ft	1300		400
Q_s in gpm/100 ft	400		----
Relief Wells			
i_a (toe of existing berm)	0.6		0.6
h_a in ft	4.2		4.2
Computed spacing in ft	175		150
Recommended spacing	150		150-100*
$\overline{W}$ in ft	100		100
El top of "extra" screen	----		----
El top of design screen	180		180
Depth of well in ft	120		120
Q_w in gpm	660		690
Standpipe ht in ft	1.5		1.5
H_a in ft	3.8		----
Seepage Berm**			
Type	Sand	Clay	----
i_o (toe of existing berm)	0.6	0.6	----
i_1 (toe of new berm)	0.8	0.8	----
h_a in ft (toe of new berm)	5.6	5.6	----
Computed X in ft†	610	1235	----
Computed width of berm extention in ft	410	1035	----
Computed t in ft††	4.0	6.9	----
Recommended X in ft†	400	400	----
Recommended approx t in ft††	4.2**	4.2**	----
Distance from center line levee to berm crown	500	500	
El berm crown	199.0	199.0	----
Berm slope	1 on 75	1 on 75	----
h_a - t in ft	4.8	4.8	----
Q_s in gpm/100 ft	425	425	----

* Between about sta 23/25 and 23/29 the well spacing would be gradually reduced from 150 to 100 ft.

** Above el 197.5.

† Measured from toe of levee proper.

†† Measured from or at toe of existing berm.

are made to accurately delineate the edge and thickness of the slough. However, the elevation of the surface of the berm extension should be the same as given in table 43.

Trotters 51, Mississippi

854. The Trotters 51 site has been subdivided into four reaches in designing control measures.

Sta 50/0 to 50/26

855. Before seepage control measures can be designed for this reach, additional borings are required to better delineate the character and thickness of the top stratum along and landward of the levee toe.

Sta 50/26 to 51/0

856. The best method of controlling seepage in this reach would be to backfill the sublevee basin to el 184 and install a line of relief wells about 90 ft deep on 150-ft centers (see table 44). Material from the sublevees could be used to fill the basins.

857. In lieu of relief wells, an extension could be added to the present landside berm to tie into the thick natural top stratum about 700 ft from the levee center line, or an extension of 200 to 300 ft. Extending the present berm without tieing into the clay-filled channel immediately landward would only aggravate the underseepage problem.

858. Construction of an impervious or random berm extension to tie into the thick clays landward will create a practically impervious top stratum 1150 ft wide from the toe of the existing berm. In computing the thickness of the berm extension it was assumed that the nearest point of seepage exit would be at the near edge of the thinner top stratum 1500 ft from the levee center line. The distance from this point to the source of seepage (s') would be 2000 ft. The effective seepage exit (x'_3) for the top stratum landward of this point was estimated to be about 1000 ft. On the basis of these assumptions and values, the head, at project flood stage, at the toe of the existing berm would be 20 ft above el 181. With $z_t = 8.0$ ft, t would have to equal 9.5 ft, or the top of the berm

Table 44

Summary of Designs of Seepage Control Measures

Trotters 51, Miss., Site

Item	Stationing of Reach* 50/26-51/0	52/0-52/45
Design Factors		
h_o in ft	18.0	11.1
H in ft	25.7	23.7
s in ft	850	850
x_3 in ft	2000	750
z_t in ft	8.0	5.0
d in ft	100	100
k_f in 10^{-4} cm/sec	1000	1000
Avg ground	183.0	185.0
L_3 in ft	1600	1300
Q_s in gpm/100 ft	235	228
Relief Wells		
i_a	0.6	0.6
h_a and H_m in ft	4.8	3.0
Computed spacing in ft	145	100
Recommended spacing	150	100
$\bar{W}$ in ft	60	50
El top of "extra" screen	----	----
El top of design screen	155	160
Depth of well in ft	90	85
Q_w in gpm	520	330
Standpipe ht in ft	2.0	1.2
Seepage Berm		
Type	Random	Sl Sd
i_o (toe of existing berm)	0.6	0.6
i_1 (toe of new berm)	0.8	0.8
h_a in ft (toe of new berm)	----	4.0
Computed X in ft**	----	1200
Computed width of berm extension in ft†	----	1000
Computed t in ft†	9.5††	6.4
Recommended X in ft**	250 to 350	300
Recommended approx t in ft†	11.5††	3.3
Distance from center line levee to berm crown	600 to 700	450
El berm crown	189.0	187.0
Berm slope	1 on 80	1 on 75
h'_o - t in ft†	6.9	2.5
Q_s in gpm/100 ft	125	310

* Data are not sufficient to design seepage control measures between sta 50/0 and 50/26, and 52/45 and 53/5.

** Measured from toe of levee proper.

† Measured from or at toe of existing berm.

†† Above el 181.0.

extension would have to be at el 190.5. To determine the distance to the crown of the berm extension, additional borings should be made to locate the edge of thick clays in the channel filling landward of the levee. The extension should overlap the edge of the thick clay by 50 ft. It is estimated that such an extension would contain about 8000 to 10,000 cu yd per 100 ft of levee.

859. If the berm extension were constructed of sand it would have the same width as mentioned above but its thickness could be reduced by 1 or 2 ft.

Sta 52/0 to 52/45

860. This reach is basically not as critical with respect to underseepage as that from sta 50/26 to 51/0 because the top stratum is more uniform and geological features do not concentrate the seepage to the same extent. However, critical uplift pressures are still likely to develop. As the top stratum along this reach is relatively thin, the required spacing for a system of relief wells is 100 ft.

861. In lieu of relief wells, an extension to the present landside seepage berm could be built, although the computed width of a semipervious berm extension that would reduce substratum pressures at the toe of the extension to tolerable amounts would be about 1000 ft. However, a berm extension of 100 ft in width (total berm width = 300 ft) is considered adequate for this reach of levee because the reach withstood a river stage within 5 ft of the project flood stage in 1937 and because the top stratum landward of the levee is relatively thin and uniform, and the maximum excess pressure that can develop is low. Such a berm extension would contain about 1200 cu yd of material per 100 ft of levee. The above design values are based on borings made along piezometer line H and geological interpretation. Additional borings would have to be made before the design of any control measure could be finalized.

Sta 52/45 to 53/55

862. The only additional seepage control measure recommended at present is the filling of the drainage ditch or construction of a gated structure at the end of the drainage ditch to insure impoundment of water up to natural ground at about el 183 during high river stages.

Trotters 54, Mississippi

863. At Trotters 54, it is considered that either the existing relief well system should be extended downstream or the landside berm downstream of the wells should be widened. For comparative purposes designs for extension of both the relief wells and the existing berm are given in table 45.

864. The required spacing for relief wells is 125 ft. These wells would start 125 ft downstream from well 30 and continue to about sta 54/20. The screens for the new wells would be set at about the same elevations as those of the existing wells.

865. An extension of the landside berm would have to be built of sand dredged from the river or of clay borrowed riverward of the riverside edge of the existing borrow pits; semipervious materials are not available at the site. As a sand extension is preferable, the design for this type berm is given in table 45. The computed width of a sand berm extension is 660 ft. However, a 200-ft extension (total berm width = 400 ft) is considered adequate for seepage control. The elevations of the berm crown and slope are given in table 45. The estimated yardage in the berm extension is about 2700 cu yd of sand per 100 ft of levee. In the design of the berm extension it was assumed that the landside drainage ditch would be filled with berm material and any new landside drainage ditch would not be excavated closer than 200 or 300 ft landward of the berm extension.

866. In lieu of the above measures, a compacted silt or clay blanket 3 ft thick could be placed in the riverside borrow pits and the landside drainage ditch backfilled. These measures would reduce substratum pressures and the severity of seepage at the project flood to less than that observed at the crest of the 1950 high water. It is estimated that about 9000 cu yd of material would be required for this blanket per 100 ft of levee. This material could be obtained by degrading the abandoned levee and from shallow borrow landward of the existing borrow pit. In order to promote willow growth in the blanketed area, the existing borrow pits should be provided with proper drainage.

Table 45

Summary of Designs of Seepage Control Measures

Trotters 54, Miss., Site

Item	Stationing of Reach 54/8	Stationing of Reach 54/20*
Design Factors		
h_o in ft	12.6	13.7
H in ft	27.7	27.7
s in ft	900	900
x_3 in ft	750	880
z_t in ft	6.5	9.0
d in ft	90	90
k_f in 10^{-4} cm/sec	1250	1250
Avg ground	180.0	180.0
L_3 in ft	∞	∞
Q_s in gpm/100 ft	286	286
Riverside Blanket		
Case		I
Type		Clay
i_a (toe of existing berm)		0.73
h_a in ft		6.3
x_r in ft		2350
L_B in ft		900
z_{bR} in ft		0
k_{bR} in 10^{-4} cm/sec		----
k_B x 10^{-4} cm/sec		**
Computed z_B in ft		**
Recommended z_B in ft		3.0
Q_s in gpm/100 ft		140
Relief Wells		
i_a	0.6	
h_a and H_m in ft	3.9	
Computed spacing in ft	125	
Recommended spacing	125	
$\overline{W}$ in ft	45	
El top of "extra" screen	----	
El top of design screen	130	
Depth of well in ft	95	
Q_v in gpm	500	
Standpipe ht in ft	1.5	
[illegible]		
Type		Sand
i_o (toe of existing berm)		0.6
i_3 (toe of new berm)		0.8
h_a in ft (toe of new berm)		7.2
Computed X in ft†		860
Computed width of berm extension in ft††		660
Computed t in ft††		6.6
Recommended X in ft†		400
Recommended approx t in ft††		4.7
Distance from center line levee to berm crown		550
El berm crown		182.0
Berm slope		1 on 75
h'_o - t in ft††		6.2
Q_s in gpm/100 ft		350

* Assuming landside ditch would be backfilled.

** It was assumed that adding 3 ft of blanket in the borrow pit would result in a blanket 5 ft thick with a permeability of 0.01 x 10^{-4} cm/sec from the levee to the river.

† Measured from toe of levee proper.

†† Measured from or at toe of existing berm.

Stovall, Mississippi

867. Relief wells are considered most applicable control measures at Stovall because of the clay-filled channels and irregular top stratum landward of the levee and high substratum pressures that develop during high water.

868. Although recommended well spacings are given in table 46 (a = 175 to 275 ft), the actual location of the wells should be based both on geological features and computed spacings. The wells should be located at the edges of and between clay-filled swales where seepage is most likely to concentrate at the stations listed on the following page.

Table 46

Summary of Designs of Seepage Control Measures

Stovall, Miss., Site

Item	Stationing of Reach				
	77/26-77/36	77/36-77/44	77/44-78/5	78/5-78/13	78/13-78/21
Design Factors					
h_o in ft	13.1	15.5	15.2	14.8	15.7
H in ft	28.8	29.8	29.8	31.3	31.3
s in ft	1200	1000	800	1000	1000
x_3 in ft	1500	1100	850	950	1000
z_t in ft	12.0	10.5	12.5	10.5	18.0
d in ft	40	40	40	40	40
k_f in 10^{-4} cm/sec	2500	2500	2500	2500	2500
Avg ground	165.5	164.5	164.5	163.0	163.0
L_3 in ft	400	400	900	1300	1500
Q_s in gpm/100 ft	----	200	280	----	----
Relief Wells					
i_a	0.6	0.6	0.6	0.6	0.6
h_a in ft	7.2	6.3	7.5	6.3	10.8
Computed spacing in ft	310	220	240	210	>300
Recommended spacing	275	175	225	200	250
$\bar{W}$ in ft	35	35	30	20	20
El top of "extra" screen	130	130	125	----	----
El top of design screen	120	120	120	115	115
Depth of well in ft	70	70	70	70	70
Q_w in gpm	620	525	730	590	680
H_m in ft	6.7	5.4	7.2	5.9	7.2
Standpipe ht in ft	2.5	2.5	2.5	2.5	2.5

77/26+75	77/45+20	78/8+70
77/29+50	77/47+45	78/10+70
77/32+25	77/49+70	78/12+70
77/35+75	78/2+20	78/15+20
77/37+50	78/4+45	78/17+70
77/39+25	78/6+70	78/20+20
77/41+00		
77/42+75		

869. The recommended well spacing and penetration of well screens at Stovall are somewhat more conservative than those obtained in the computations because the past seepage record of the site indicates that the design of any control measure should be conservative.

Farrell, Mississippi

870. Between sta 81/10 and 81/30, the landside top stratum is relatively thin and uniform and it is believed that a landside seepage berm represents the most satisfactory control measure. If the clay seam at el 140 is continuous along this reach, a 100-ft-wide extension to the present landside berm would be adequate. The extension should consist of sand, as semipervious materials are not available at the site. It is estimated that about 750 to 1000 cu yd of sand would be required for the extension per 100 ft of levee. However, rather than borrow the additional material required for a berm extension, the necessary material can be obtained by degrading the existing berm and widening it to the dimensions indicated in table 47. It should be noted that the extended berm would be only about 50 ft wider than the existing berm and considerably narrower than the extension recommended if the existing berm is not reshaped. The need for only a small increase in width if the berm is reshaped is due to the fact that the new berm will be sufficiently thin to permit seepage to emerge through it, whereas the existing berm is too thick to permit emergence of seepage. In the design it was assumed that the reshaped berm would act, in effect, as a silty sand berm, and that the landside drainage ditch would be backfilled with excess material removed from the existing berm. It should also be noted that if the clay seam at el 140 is continuous, the combined thickness of soil above the

Table 47

Summary of Designs of Seepage Control Measures

Farrell, Miss., Site

Item	Stationing of Reach: 81/10-81/30	81/10-31/30	81/10-81/30*	81/30-81/42	81/42-81/48	81/48-82/13
Design Factors						
h_o in ft	5.0	7.0**	12.1	14.7	16.9	13.7
H in ft	26.5	26.5	26.5	28.1	28.1	26.1
s in ft	575†	375**	575†	1000	1000	900
x_3 in ft	135	135	485	1100	1500	1000
z_t in ft	3.0	3.0	3.0	7.0	16.0	10.0
d in ft	20††	20††	70	70	70	70
k_f in 10^{-4} cm/sec	300††	300††	1000	1000	1000	'00
Avg ground	165.6‡	165.6‡	165.6‡	164.0	164.0	166.0
L_3 in ft	∞	∞	∞	∞	∞	∞
Q_s in gpm/100 ft	32	----	----	----	----	----
Relief Wells						
i_a (toe of existing berm)			0.6	0.6	0.6	0.6
h_a in ft			1.8	4.2	9.6	6.0
Computed spacing in ft			45	140	>300	220
Recommended spacing in ft			75	150	325	220
$\bar{W}$ in ft			40	40	40	40
El top of "extra" screen			----	----	----	----
El top of design screen			135	125	125	135
Depth of well in ft			70	80	80	70
Q_w in gpm			300	315	520	600
Standpipe ht in ft			0.5	1.5	4.0	2.0
H_m in ft			> 2.4	4.5	7.9	6.0
Seepage Berm						
Type	Sand Extension	Si Sd‡‡	Sand Extension§			
i_o (toe of levee or existing berm)	0.60†	0.5**	0.60†			
i_1 (toe of new berm)	0.80	0.80	0.80			
h_a in ft (toe of new berm)	2.4	2.4	2.4			
Computed X in ft**	310	210	1040			
Computed width of berm extension in ft*	110	----	840			
Computed t in ft	2.3†	4.7**	6.4†			
Recommended X in ft**	300	210	400			
Recommended approx t in ft§§	3.0	5.2	4.3			
Distance from center line levee to berm crown	475	400	575			
El berm crown	169.0	169.0	169.0			
Berm slope	1 on 50	1 on 50	1 on 60			
h'_o - t in ft	2.5	3.4	3.7			
Q_s in gpm/100 ft	29	37	210			

* Recommended measures if clay seam at el 140 is not continuous along the reach.
** Measured at or from toe of levee proper.
† Measured at or from toe of existing berm.
†† Thickness and permeability of aquifer above clay seam at el 140.
‡ For design of berms average ground taken as el 168.0, as landside low area would be covered by berm and landside ditch would be backfilled.
‡‡ Total berm if existing berm is degraded.
§ Design of berm based on z_t = 6.0 ft and x_3 = 690 ft, i.e., conditions at toe of computed berm.
§§ Above el 168.0.

clay seam is considered sufficient to withstand substratum pressures developing at the base of the seam, and control measures in addition to those described above are not necessary.

871. Should the clay seam not be continuous along the reach, the effective aquifer thickness is then about 70 ft instead of 20 ft and the computed width of a sand berm extension would be about 800 ft. However, it is believed that a 200-ft-wide extension, or a total berm width of 400 ft, would suffice for this condition.

872. Relief wells could be installed in lieu of the berm extension. The computed spacing required for the wells is about 45 ft if the clay seam is not present; however, it is believed that wells spaced on about 75-ft centers would intercept sufficient seepage to insure the safety of the levee against piping. The continuity of the clay seam at el 140 should be checked along the reach of levee between sta 81/10 and 81/30 before the design of any seepage control measure is finalized.

873. Downstream from sta 81/30 the top stratum is comprised of point bar ridge and swale deposits. A line of relief wells about 70 to 80 ft deep would be the most satisfactory method of seepage control along this reach. The suggested well spacings and elevations of well screens in this reach are given in table 47. For most efficient seepage control, the wells should be spaced, with consideration given to the geologic and topographic features, approximately as follows

81/30+00	81/36+00	81/47+00	82/5+20
81/31+50	81/37+50	81/4[illegible]+00	82/7+40
81/33+00	81/39+00	81/51+00	82/9+60
81/34+50	81/40+50	82/0+80	82/11+80
	81/43+75	82/3+00	82/14+00

Upper Francis, Mississippi

874. As the top stratum landward of the levee is rather thick and impervious, pressure relief can be achieved most readily at Upper Francis by relief wells along the toe of the present berm. The well spacing recommended is 300 ft, as shown in table 48.

875. A riverside blanket would be of no benefit at this site as

Table 48

Summary of Designs of Seepage Control Measures

Upper Francis, Miss., Site

Item	Stationing of Reach	
	[illegible]-47	47-60
Design Factors		
h_o in ft	19.9	12.4
H in ft	25.0	27.0
s in ft	1500	1700
x_3 in ft	1950	1450
z_t in ft	16	12
d in ft	125	115
k_f in 10^{-4} cm/sec	1400	1400
Avg ground	160.0	158.0
L_3 in ft	∞	5000
Q_s in gpm/100 ft	185	200
Relief Wells		
i_a	0.6	0.6
h_a in ft	9.6	7.2
Computed spacing in ft	>300	>300
Recommended spacing	300	300
$\overline{W}$ in ft	75	70
El top of "extra" screen	----	110
El top of design screen	110	100
Depth of well in ft	128	128
Q_w in gpm	850	780
Standpipe ht in ft	3.0	3.0
H_m in ft	6.0*	5.5**

* Includes an extra 0.7-ft friction loss in well because of high Q_w.

** Includes an extra 0.5-ft friction loss in well because of high Q_w.

the existing borrow pits are presently blanketed with an adequate thickness of clay. The primary source of seepage at Upper Francis is believed to be through the natural top stratum riverward of the borrow pit.

876. An extension to the existing seepage berm is not considered practical because of the comparatively thick and impervious top stratum which tends to cause development of high uplift pressures for considerable distances landward of the levee. Extension of the present berm would thicken the top stratum but would not provide pressure relief. (In areas where the top stratum is relatively thick but critical uplift pressures still develop, the most efficient control measure is considered to be relief wells.)

Lower Francis, Mississippi

877. Between sta 140 and 150 where the clay-filled channel is close to the levee and tends to concentrate seepage, it is recommended that a line of relief wells about 100 ft deep be installed along the toe of the berm. The wells should be spaced on 150- to 75-ft centers at locations given below. The spacing would be progressively decreased as the distance from the levee to the clay-filled channel becomes less and seepage becomes more concentrated.

140+50	147+50
142+00	148+50
143+50	149+25
145+00	150+00
146+25	

878. Upstream from sta 140+50 the existing berm is thicker than necessary but is considered too narrow. Therefore it is recommended that this berm be degraded to the dimensions indicated in table 49. The existing berm contains sufficient material for the wider and thinner berm and no other borrow is considered necessary.

879. For comparative purposes, dimensions of an extension to the existing berm, if the berm is not reshaped, are given in table 49. Such an extension would contain about 2600 cu yd of sand per 100 ft of levee. The sand probably would have to be dredged from the river. As this

Table 49

Summary of Designs of Seepage Control Measures

Lower Francis, Miss., Site

Item	Stationing of Reach				
	114/0-122/0	122/0-140/0	140/0-150/0	114/0-122/0	122/0-140/0
Design Factors					
h_o in ft	8.7*	9.0*	10.2**	7.3**	7.7**
H in ft	27.1	28.6	28.6	27.1	28.6
s in ft	700*	800*	1100**	900**	1000**
x_3 in ft	330	370	610	330	370
z_t in ft	3.5	4.5	6.0	3.5	4.5
d in ft	135	135	135	135	135
k_f in 10^{-4} cm/sec	1600	1600	1600	1600	1600
Avg ground	155.5	154.0	154.0	155.5	154.0
L_3 in ft	1000**	800**	600**	1000**	800**
Q_s in gpm/100 ft	----	790	675	----	790
Relief Wells					
i_a			0.6		
h_a and H_m in ft			3.6		
Computed spacing in ft			140		
Recommended spacing			135†		
$\overline{W}$ in ft			80		
El top of "extra" screen			----		
El top of design screen			135		
Depth of well in ft			100		
Q_w in gpm			790		
Standpipe ht in ft			1.5		
Seepage Berm	Condition 1††			Condition 2‡	
Type	Sand	Sand	----	Sand	Sand
i_o (toe of levee or existing berm)	0.5*	0.5*	----	0.6**	0.6**
i_1 (toe of new berm)	0.8*	0.8	----	0.8	0.8
h_a in ft (toe of new berm)	2.8	3.6	----	2.8	3.6
Computed X in ft*	525	450	----	635	560
Computed width of berm extension in ft	----	----	----	435**	360**
Computed t in ft	5.7*	5.5*	----	4.1**	3.7**
Recommended X in ft*	400	400	----	400	400
Recommended approx t in ft	7.2*	7.7*	----	4.2**	4.7**
Distance from center line levee to berm crown	600	600	----	600	600
El berm crown	157.0	156.0	----	157.0	156.0
Berm slope	1 on 70	1 on 70	----	1 on 75	1 on 75
$h'_o - t$ in ft	3.8*	4.2*	----	3.0**	3.1**
Q_s in gpm/100 ft	715	735	----	715	735

* Measured from or at toe of levee proper.
** Measured from or at toe of existing berm.
† Between sta 140+50 and 150+00 the well spacing should be reduced gradually from 150 ft to 75 ft.
†† Required berm if existing berm is partially degraded.
‡ Sand extension if existing berm is not reshaped.

extension requires procurement of additional material it is not recommended.

880. Summarizing, it is believed that 9 relief wells should be installed along the toe of the existing berm between about sta 140 and 150, and that upstream from sta 140 the existing berm should be degraded and reshaped as indicated in table 49.

Bolivar, Mississippi

881. Additional control measures recommended at Bolivar are a line of relief wells along the levee toe or a landside berm. Designs of these measures are summarized in table 50.

882. To insure adequate relief of pressure landward of the levee, a line of relief wells spaced on 75-ft centers would be required along the levee toe between sta 2190 and 2220. The wells should be about 100 ft deep.

883. In lieu of relief wells, a landside berm could be constructed. Although a berm could be built of clays and silts obtained at above el 140 at the riverside edge of the existing borrow pits, such a berm would require considerably more material than a sand berm and would not be as efficient. Therefore, a berm built of sand dredged from the river would appear to be more practical. The dimensions of the sand berm are given in table 50. The crown of the berm would be located along the landside sublevee; therefore, construction of the berm would consist essentially of filling the existing sublevee basins, thus reinforcing the top stratum. The berm would have a width of about 250 to 300 ft and contain about 5500 cu yd of material per 100 ft of levee. The berm probably could be placed by hydraulic methods and the existing sublevee used as a retaining dike. The sublevee would later be degraded to the top of the berm.

884. Even if a landside berm is constructed at Bolivar, three relief wells on about 100-ft centers should be placed along the toe of the new berm between sta 2193+50 and 2195+50, because a thick clay-filled channel would create a potentially hazardous situation along the toe of the new berm between these levee stations.

Table 50

Summary of Designs of Seepage Control Measures

Bolivar, Miss., Site

Item	Stationing of Reach 2190-2205	Stationing of Reach 2205-2220
Design Factors		
h_o in ft	12.2	9.8
H in ft	26.2	26.2
s in ft	500	500
x_3 in ft	350	300
z_t in ft	7.0	6.0
d in ft	90	90
k_f in 10^{-4} cm/sec	1200	1200
Avg ground or tailwater	141.0	141.0
L_3 in ft	500	-
Q_s in gpm/100 ft	520	620
Relief Wells		
i_a	0.5	0.5
h_a in ft	3.5	3.0
Computed spacing in ft	75	65
Recommended spacing	75	75
$\bar{W}$ in ft	55	55
El top of "extra" screen	----	----
El top of design screen	95	95
Depth of well in ft	100	100
Q_w in gpm	420	390
Standpipe ht in ft	1.5	1.5
H_m in ft	3.5	3.4
Seepage Berm		
Type	Sand	Sand
i_o (toe of levee)	0.5	0.5
i_1 (toe of new berm)	0.8	0.8
h_a in ft (toe of new berm)	5.6	4.8
Computed X in f	290	270
Computed t in ft	5.5*	5.1*
Recommended X in ft	250 to 300	250 to 300
Recommended approx t in ft	8.5**	7.5**
Distance from center line levee to berm crown	475	475
El berm crown	142.0	142.0
Berm slope	1 on 50	1 on 50
h'_o - t in ft	5.3	4.2
Q_s in gpm/100 ft	425	485

Note: If landside berm is constructed three wells on 100-ft centers should be installed along the berm toe between sta 2193 and 2195.

* Above el 141.0.

** Above ground surface at toe of levee.

Eutaw, Mississippi

885. It is difficult to design with exactitude control measures for the levee at Eutaw, because of irregularities in ground elevation and the type and thickness of top stratum landward of the seepage berm. The most practical solution appears to be the construction of sublevees across the old slough at sta 2820 and 2900 and on the landside high bank of the slough to an elevation (140) that will permit impounding water up to el 138.0 (table 51). Impounding water to this elevation would reduce the estimated maximum net uplift beneath the slough to about 7 or 8 ft, which should be safe. The sublevee basin should be provided with the usual control gates and paved overflow spillways since it is estimated that about 25 gpm of seepage water will enter the sublevee basin per 100 ft of levee at the project flood.

886. A potentially critical underseepage area would still exist upstream of sta 2840 (upstream end of existing berm) as the sublevee would not impound water over the area, and a very thin and narrow reach of top stratum exists between the levee toe and much thicker landward top stratum. Either a relief well system or a berm combined with the sublevee described above is considered necessary along this reach.

887. The dimensions of a semipervious berm are given in table 51. This berm would cover the thin top stratum between the levee and landside slough and was designed to protect the thin top stratum only. It should be noted that this berm is somewhat thinner than the existing berm downstream from sta 2840; however, the latter berm is thicker than necessary.

888. In lieu of a berm, relief wells about 88 ft deep on 100-ft centers could be installed along the toe of the levee upstream from sta 2840. It is pointed out that soils data for the reach upstream of sta 2840 are somewhat limited; therefore, these control measures should not be installed until additional soils data and geological information are obtained and the above designs are reviewed.

Table 51

Summary of Designs of Seepage Control Measures

Eutaw, Miss., Site

Item	Stationing of Reach: Upstream of 2640	Stationing of Reach: 2640-2660
Design Factors		
h_o in ft	10.0	11.2
H in ft	25.1	28.1
s in ft	1500	1500
x_3 in ft	1000	1000
z_t in ft	4.5	13.0
d in ft	70	70
k_f in 10^{-4} cm/sec	1100	1100
Avg ground	138.0*	135.0
L_3 in ft	150	*
Q_s in gpm/100 ft	----	132
Relief Wells		
i_a	0.5	----
h_a in ft	2.3	----
Computed spacing in ft	120	----
Recommended spacing	100	----
$\bar{W}$ in ft	40	----
El top of "extra" screen	----	----
El top of design screen	90	----
Depth of well in ft	68	----
Q_w in gpm	155	----
Standpipe ht in ft	1.0	----
H_w in ft	2.0	----
Seepage Berm		
Type	S1 Sd	Sublevee**
i_o (toe of levee or existing berm)	0.5†	0.6††
i_1 (toe of new berm)	0.6	0.8
h_a in ft (toe of new berm)	2.9	7.2‡
Computed X in ft	‡‡	80††
Computed t in ft	‡‡	3.6††
Recommended X in ft	200†	200††
Recommended approx t in ft	6.7†	3.0
Distance from center line levee to berm crown	350	550±
El berm crown	142.0	138.0
Berm slope	1 on 75	----
h'_o - t in ft	3.3†	8.9
Q_s in gpm/100 ft	125	125
Q'_s in gpm/100 ft	25	25

Note: Berm should extend over near edge of landside slough.
* Tailwater with sublevee basin full.
** Also applicable upstream from sta 2640.
† Measured from or at toe of existing levee.
†† Measured from or at toe of existing berm.
‡ Beneath LS slough.
‡‡ Design formulas not applicable as L_3 is only 150 ft.

L'Argent, Louisiana

889. The most practical seepage control measure at L'Argent is considered to be a line of relief wells along the levee toe to reduce excessive landside substratum pressures. The computed spacing for the wells is somewhat greater than 300 ft (see table 52), but as the maximum

Table 52

Summary of Designs of Seepage Control Measures

L'Argent, La., Site

Item	Stationing of Reach 3526-3552
Design Factors	
h_o in ft	20.0
H in ft	31.0
s in ft	3000
x_3 in ft	5500
z_t in ft	15.5
d in ft	120
k_f in 10^{-4} cm/sec	400
Avg ground	59.0
L_3 in ft	1340
Q_s in gpm/100 ft	30
Relief Wells	
i_a	0.5
h_a in ft	7.8
Computed spacing in ft	>300
Recommended spacing	300
$\bar{W}$ in ft	70
El top of "extra" screen	----
El top of design screen	30
Depth of well in ft	100
Q_w in gpm	155
H_m in ft	4.3
Standpipe ht in ft	3

practicable spacing for relief wells is considered to be about 300 ft, this spacing is recommended. The elevations of the well screens and estimated well flows are given in table 52.

890. A seepage berm is not considered a practical method of reducing substratum pressures along this reach of levee because of the thickness and imperviousness of the top stratum.

891. Data available are not adequate to design seepage control measures for the levee downstream of sta 3552.

Hole-in-the-Wall, Louisiana

892. It is believed that either the present seepage berm should be widened or a line of relief wells installed along the toe of the berm to insure the safety of the levee at Hole-in-the-Wall.

893. The computed well spacing along this site varies from about 125 to 250 ft (see table 53). The wells should be about 105 ft deep.

894. The present landside seepage berm could be extended to provide adequate protection to the levee. The computed required extension is about 350 to 430 ft (see table 53). However, it is believed that a 125-ft extension, or total berm width of 350 ft, would be adequate. The extension should consist of silty sand or sand obtained riverward of the levee and would contain about 1000 cu yd of material per 100 ft of levee. Material for the berm extension should be obtained riverward of the existing borrow pit; otherwise the distance to the source of seepage may be decreased and the control measures recommended would not be adequate.

Table 53

Summary of Designs of Seepage Control Measures

Hole-in-the-Wall, La., Site

Item	Stationing of Reach 3603-3606	3606-3623	3623-3632
Design Factors			
h_o in ft	8.4	6.4	7.3
H in ft	24.5	23.5	23.5
s in ft	1200	1200	1200
x_3 in ft	625	450	535
z_t in ft	7.5	4.0	5.5
d in ft	130	130	130
k_f in 10^{-4} cm/sec	500	500	500
Avg ground	65.0	66.0	66.0
L_3 in ft	∞	∞	∞
Q_s in gpm/100 ft	----	115	----
Relief Wells			
i_a	0.6	0.6	0.6
h_a in ft	4.5	2.4	3.3
Computed spacing in ft	270	130	205
Recommended spacing	250	125	200
$\bar{W}$ in ft	70	70	70
El top of "extra" screen	----	----	----
El top of design screen	30	30	30
Depth of well in ft	105	105	105
Q_w in gpm	270	160	215
Standpipe ht in ft	1.5	1.0	1.5
H_m in ft	4.4	2.3	3.3
Seepage Berm			
Type	Si Sd	Si Sd	Si Sd
i_o (toe of existing berm)	0.6	0.6	0.6
i_1 (toe of new berm)	0.8	0.8	0.8
h_a in ft (toe of new berm)	6.0	3.2	4.4
Computed X in ft*	630	655	575
Computed width of berm extension in ft**	405	430	350
Computed t in ft**	4.1	3.3	2.9
Recommended X in ft*	350	350	350
Recommended approx t in ft	3.7	2.7	2.7
Distance from center line levee to berm crown	475	475	475
El berm crown	67.0	67.0	67.0
Berm slope	1 on 75	1 on 75	1 on 75
h'_o - t in ft	4.0	2.5	3.5
Q_s in gpm/100 ft	140	150	140

* Measured at or from toe of levee proper.
** Measured at or from toe of existing berm.

PART X: SUMMARY OF RESULTS

895. The results of the field, laboratory and office studies comprising the investigation are summarized in the following paragraphs.

Geology of the Lower Mississippi River Valley and Its Influence on Underseepage

896. The alluvial valley of the Lower Mississippi River is filled with a series of sandy gravels, sands, silts, and clays that can be grouped into two broad units: (a) the sand and gravel substratum, and (b) the fine-grained top stratum. The alluvial materials are generally underlain by relatively impervious Tertiary marine deposits. Above Baton Rouge, sand substratum is quite pervious and ranges in thickness from about 75 to 150 ft. The top stratum usually is about 5 to 20 ft thick except where backswamp deposits or clay-filled channels exist. Geological studies showed a definite correlation between the distribution of alluvial deposits of sand, silt, and clay and the location and occurrence of underseepage and sand boils. The actual location of seepage and sand boils is commonly influenced by: (a) configuration of geological features, such as swale fillings and clay plugs, and their relation to the levee; (b) characteristics and thickness of the landside top stratum; (c) cracks, fissures, holes, ditches, etc., formed by natural causes, man, or animals.

Occurrence and Analysis of Underseepage

Development of underseepage and sand boils

897. Whenever a levee is subjected to a differential hydrostatic head of water as a result of river stages higher than the surrounding land, seepage enters the pervious substratum through the bed of the river, riverside borrow pits, and/or the riverside top stratum, and creates an artesian head and hydraulic gradient in the sand stratum under the levee. This gradient causes a flow of seepage beneath the levee and

the development of excess pressures landward thereof. If the hydrostatic pressure in the pervious substratum landward of the levee becomes greater than the submerged weight of the top stratum, the excess pressure will cause heaving of the top blanket, or will cause it to rupture at one or more weak spots with a resulting concentration of seepage flow in the form of sand boils.

898. In nature, seepage usually concentrates along the landside toe of the levee, at thin or weak spots in the top stratum, and adjacent to clay-filled swales or channels. Where seepage is concentrated to the extent that turbulent flow is created, the flow will cause erosion in the top stratum and development of a channel down into the underlying silts and fine sands which frequently exist immediately beneath the top stratum. Not only are excessively high substratum pressures and underseepage a hazard from the standpoint of underground erosion but they may also saturate and reduce the stability of the landside slope of a levee.

899. A number of levee crevasses have occurred as a result of sand boils, and it is possible that other crevasses would have occurred as the result of underseepage had not determined efforts been made to hold the levees.

900. Whether or not a specific levee will be crevassed as a result of critical substratum pressures and concentrated seepage in the form of sand boils or piping is practically impossible to predict. However, excessive substratum pressures and active sand boils are potential hazards to the safety of a levee. The partial formation of pipes under a levee can result in progressive collapse of the soil and accelerated erosion which may ultimately cause a blowout unless control measures are constructed.

Computation of seepage flow and substratum pressures

901. Seepage flow and hydrostatic heads landward of a levee can be estimated from seepage formulas and/or piezometric data, and a knowledge of the top stratum characteristics both riverward and landward of the levee, and of the pervious substratum. However, the accuracy of results obtained from such formulas is dependent on the applicability of the

formula to the condition being analyzed, the uniformity of soil conditions, and evaluation of the various factors involved in the computations.

902. Methods of determining the factors and characteristics of the foundation necessary for making seepage analyses include surveys, field explorations, laboratory tests, field pumping tests, and piezometers. Methods of arriving at numerical values of these factors are summarized below.

a. Length of top stratum landward of levee. This factor is usually considered to be infinite unless changes in geology or topography limit the emergence of seepage to a definite area. The distance to such a block created by high ground or a wide clay-filled slough can be ascertained from field reconnaissance, geological studies, aerial mosaics, borings, and/or topographic maps.

b. Slope of hydraulic grade line beneath levee. This can best be determined from piezometers located beneath the levee where the seepage flow lines are essentially horizontal and the equipotential lines vertical. The hydraulic grade line as determined in the field from piezometer readings is the most reliable method for determining the effective seepage entrance and exit and is of use in computing the quantity of seepage passing beneath the levee.

c. Effective thickness and permeability of top stratum. The thickness of the top stratum both riverward and landward of a levee is usually determined by auger borings. Borings should be made to delineate the thickness and extent of any geological feature within 500 ft landward of the levee toe that may significantly affect the seepage analysis. The thickness of the top stratum in the bottom of landward ditches should also be determined. Where the thickness of the riverward top stratum has been reduced by borrow pits, the thickness of any remaining blanket should be determined by shallow borings or estimated from landside borings and the elevation of the bottom of the pits.

The vertical permeability of the landward top stratum can probably best be determined from observed hydrostatic heads beneath the landside top stratum together with seepage measurements. The permeability of the top blanket can also be computed from known characteristics of the pervious foundation and the effective seepage exit as determined from the hydraulic grade line in the pervious foundation beneath the levee.

Where borrow pits, ditches, or channels exist within 200 to 300 ft of the landside levee toe, the thickness of top stratum used in computing seepage flows and substratum pressures should be based on the thickness of the top blanket adjacent to the

ditch, unless the ditch or borrow pit is very wide. The allowable critical substratum pressure should be computed for both the thickness of the top stratum at the toe of the levee and at the bottom of the ditch.

d. Effective thickness and permeability of pervious substratum. The effective thickness of the pervious substratum, i.e., the thickness of the principal seepage-carrying sand stratum below the top stratum and above the bottom of the entrenched valley, may be determined by deep borings or a combination of shallow borings and seismic or electrical resistivity surveys. Its average horizontal permeability can best be determined by pumping tests on a well fully penetrating the pervious aquifer. The permeability of individual sand strata can be determined by measuring the well flow in the screen at the boundary of the sand stratum being tested. The average horizontal permeability of the pervious aquifer can also be determined from pumping tests on partially penetrating wells using data from piezometers located some distance from the well where the flow lines to the well are essentially horizontal. When the permeability of the pervious aquifer cannot be determined from pumping tests it can be estimated from a correlation of the effective grain size and permeability of the various sand strata.

Pumping tests show that the actual horizontal permeability of a sand stratum in the Mississippi River valley is usually two to four times greater than that indicated by laboratory permeability tests on remolded samples. The average horizontal permeability of the pervious strata beneath the levee can also be estimated from the hydraulic grade line beneath the levee and seepage passing beneath the levee.

e. Effective source of seepage entry. The best and most accurate method for determining the distance from the landside levee toe to the effective source of seepage entry is to project on a straight line the hydraulic grade line beneath the levee until it intersects the river stage producing the gradient. However, this method is not valid unless the pervious stratum beneath the levee is saturated and artesian flow is established. The distance to the effective source of seepage entry also may be estimated from seepage formulas and the characteristics of the riverside top stratum and pervious substratum.

f. Distance from landside levee toe to effective seepage exit may best be determined by projecting the hydraulic grade line beneath the levee landward until it intersects the ground surface or tailwater. It may also be estimated from seepage formulas and the characteristics of the landside top stratum and pervious substratum.

g. Critical gradient. The critical gradient required to cause sand boils, or heaving or flotation of the top stratum, is

the ratio of the unit weight of submerged soil comprising the top stratum to the unit weight of water. The theoretical critical gradient for most top stratum soils is about 0.80 to 0.85. The critical gradient required to cause sand boils can best be determined in the field by noting the hydrostatic head beneath the top stratum at the time sand boils first appear.

Investigation of Underseepage at Piezometer Sites

903. Sites were selected for study where a maximum amount of subsurface exploration already had been made and where a wide range of representative geological and top stratum conditions exists. At some of the sites no underseepage had occurred and at others seepage and sand boils had been a serious problem during the 1937 high water.

904. The analyses of seepage and piezometric data for each site included determination of the: distance to the Mississippi River and length of the landside top stratum; condition and size of riverside borrow pits; effective thickness and permeability of pervious substratum and landside top stratum; effective seepage source and exit; seepage flow beneath the levee; substratum pressures landward of the levee; and hydraulic gradients beneath and landward of the levee during high water. Predictions were made as to substratum pressures and hydraulic gradients to be expected at project flood stages. Such predictions were based on theoretical formulas, extrapolation of observed data, and borrow pit and seepage control measures as they existed in 1950, and led to recommendations of additional seepage control measures at the following 13 sites of the 16 studied: Caruthersville, Gammon, Commerce, Trotters 51, Trotters 54, Stovall, Farrell, Upper Francis, Lower Francis, Bolivar, Eutaw, L'Argent, and Hole-in-the-Wall.

Evaluation of Data from Piezometer Sites

Characteristics of riverside top stratum

905. Source of seepage and effective length of riverside blanket.

Of the 15 sites at which sufficient piezometric data were available for analysis, the source of seepage at the crest of the 1950 high water was located in the riverside borrow pits except where the borrow pits were blanketed with a thick layer of clay. The distance to the effective source of seepage entry generally ranged from about 600 to 3000 ft. The corresponding effective length of riverside blanket ranged from about 200 to 2800 ft.

906. At L'Argent and Baton Rouge, where the riverside borrow pits are blanketed with clay 15 to 20 ft thick, the effective source of seepage was located at the bank of the Mississippi River. At the other sites where the thickness of the riverside blanket ranged from about 0 to 5 ft, the source of seepage generally was in the riverside borrow pits.

907. Generally, the effective length of riverside blankets tended to increase as the blanket material in the borrow pits graded from silty sand to clay, and also as a given type of blanket increased in thickness. A top stratum of silty sand is not very effective, as the length of this type riverside top stratum was about the same as at sites where the substratum sands were exposed in the borrow pits. Where the riverside blanket consisted of clay 10 to 15 ft thick, very little seepage penetrated through the blanket.

908. These studies indicate that the underseepage problem along the Lower Mississippi River levees has been aggravated by more or less complete removal of the riverside top blanket along certain reaches of levee as the result of borrow operations for construction of the levee. Where feasible, borrow operations riverward of a levee should be controlled so as not to expose the underlying sand aquifer.

909. Permeability. For a given material, the permeability of the riverside top stratum k_{bR} generally tends to decrease as the thickness of top stratum increases, particularly for clay top strata. Values of k_{bR} were zero at sites where thickness of the blanket equalled or exceeded 15 ft of clay as compared to about 1×10^{-4} cm per sec where the clay blanket was less than 5 ft thick. No apparent decrease in k_{bR} with increasing blanket thickness was observed at sites where the borrow pits were blanketed with silt. The average permeability of the silty blankets

was about 2.5 x 10^{-4} cm per sec; k_{bR} for silty sand blankets up to 10 ft thick averaged about 6 x 10^{-4} cm per sec.

910. Data from piezometers installed at various depths in the pervious aquifer indicate that: (a) the head immediately beneath the top stratum under the levee crown is equal to the average head on a vertical plane through the substratum sands under the crown, and (b) where there is a significant upward flow of seepage the head immediately beneath the top stratum at the landside toe of the levee is somewhat less than the average head in the sand stratum under the levee toe.

Characteristics of landside top strata

911. Effective seepage exit. Values of x_3 generally ranged from about 150 to 11,000 ft, the largest occurring at Baton Rouge where the top stratum is about 30 ft thick. The distance to the effective seepage exit was usually rather short at sites where the landside top stratum was thin and at sites where numerous sand boils developed. At sites where the exit of seepage was partially blocked as a result of landward swales or sloughs, the observed values of x_3 and h_o were larger than they would have been without the blocked exit.

912. Values of x_3 followed three basic patterns during rising river stages: (a) a constant x_3 indicated that resistance to the flow of seepage either landward or up through the natural blanket was constant for the river stages experienced; (b) a decrease in x_3 with rising river stage usually occurred when sand boils began to develop (such boils provide additional seepage outlets, thereby decreasing the resistance offered by the natural blanket to the emergence of seepage); (c) an increase in x_3 with rising river stage indicated an increase in resistance to the flow of seepage landward. (At the beginning of overbank stages, the natural water table may be low and seepage may readily flow into the resultant large volume of ground-water storage which in a sense acts as a drainage area. As the subsurface storage becomes filled, the phreatic line comes in contact with the bottom of the top stratum and seepage either has to flow toward storage areas farther landward or force its way up through the top stratum. In either case resistance to the flow of

seepage landward is increased, thereby increasing the distance to the effective seepage exit.)

913. The accuracy of x_3 as determined from piezometer readings is affected by the average ground or tailwater elevation, and therefore reconnaissance should be made during high-water periods to determine the elevation of the water in submerged areas.

914. Thickness and permeability. The thickness of the landside top stratum varied from about 4 to 30 ft; the permeability k_{bL} generally ranged from about 0.06×10^{-4} cm per sec to about 10×10^{-4} cm per sec. At sites where numerous sand boils occurred, considerably higher values of permeability were noted. Most values of k_{bL} at the crest of the 1950 high water ranged from about 0.5 to 10×10^{-4} cm per sec.

915. There was a pronounced trend for k_{bL} to decrease with an increase in z_{bL}, particularly for clayey top strata; there was a lesser tendency for k_{bL} to decrease with increasing thickness of silty top stratum. The permeability of top strata less than 10 ft thick was about the same for silt as for clay. The permeability of the top stratum landward of the levee has little relation to that which would be obtained from laboratory tests on undisturbed samples, but instead depends to a large extent on the presence and numbers of fissures, root holes, former boil holes, and other perforations in the top stratum. The effect of these perforations in clay top stratum appears to be reduced if the blanket thickness exceeds 10 ft, and greatly reduced if z_{bL} exceeds 15 ft.

916. Comparisons between k_{bR} and k_{bL} for similar blankets of similar thickness indicate that the landside blanket tends to be about two to ten times as pervious as the riverside blanket. As cracks and fissures exist on both sides of the levee, this difference is attributed to the tendency of upward seepage landside to flush out the cracks and perforations, thereby increasing the over-all permeability of the top stratum. Downward seepage through the riverside blanket tends to seal any cracks or fissures unless excessive erosion occurs.

Characteristics of pervious substratum

917. Effective thickness of the pervious substratum ranged from about

70 to 165 ft and averaged about 110 ft for the sites studied. Estimated values of k_f ranged from 400 to 2500 x 10^{-4} cm per sec. For most sites in the Memphis and Vicksburg Districts above L'Argent, La., k_f ranged from about 1000 to 1500 x 10^{-4} cm per sec. At L'Argent and sites farther downstream, k_f was estimated to be no more than about 500 x 10^{-4} cm per sec. Although it should not be inferred that k_f will always be less than 500 x 10^{-4} cm per sec in the alluvial valley of the Mississippi River below L'Argent, lower k_f values generally can be expected below L'Argent. Good agreement was obtained between values of k_f as estimated from a correlation of grain size and permeability and those determined from analyses of piezometric data and natural seepage measurements, well flow data, and pumping tests. Poor agreement was obtained between k as estimated from laboratory permeability tests and k_f obtained from piezometric, seepage, and well flow data, and/or pumping tests.

Ratio of permeability of pervious substratum to landside top stratum

918. Values of k_f/k_{bL} obtained at the piezometer sites at the crest of the 1950 high water ranged from about 100 to 2000 except at Baton Rouge where the ratio was about 8500. There was a tendency for k_f/k_{bL} to increase for clay blankets as the top stratum increased in thickness; however, there was no apparent variation in k_f/k_{bL} with z_{bL} for sites where the top stratum was predominantly silt.

Critical upward gradient

919. Upward gradients through the top stratum as measured by piezometers during the 1950 high water and the degree of seepage were:

Seepage Conditions	i
Light to no seepage	0 to 0.5
Medium seepage	0.2 to 0.6
Heavy seepage	0.4 to 0.7
Sand boils	0.5 to 0.8

The gradient required to cause sand boils varied considerably at the different sites, possibly because at sites where sand boils had developed previously only fairly low excess heads may have been needed to reactivate

these boils in 1950. At sites where no sand boils had occurred in the past, higher gradients may have been required to initiate formation of the boils, although this is difficult to ascertain because of limited data on previous seepage at the sites. From the above data, it appears that heavy seepage and sand boils should be anticipated whenever estimated upward gradients exceed 0.5 to 0.8, depending on site conditions.

Effect of natural partial cutoffs and massive clay deposits on seepage

920. An examination of piezometric gradients where natural partial cutoffs exist shows no significant drop in head across the partial cutoffs. Massive clay deposits a short distance landward of the levee toe are believed to have increased the severity of the seepage conditions which occurred during the 1937 and 1950 high waters at Trotters 51 and Stovall.

Seepage berms at piezometer sites

921. Except for the seepage berm at Gammon, berms at the other sites are of such soil types and/or thickness as to make them practically impervious. Assuming that the riverside and landside blankets remained unchanged as a result of construction of the berms, the 200-ft-wide berms typical of most sites probably decreased seepage and landward pressures by approximately 10 to 15 per cent from what would have occurred with no berm. Since borrow for most of these berms was obtained riverward of the levee, the borrow operations may have reduced the effective values of x_1 as much as the width of the berm increased L_2. If such is the case, little or no reduction in Q_s or h_o may have resulted from the berm. Construction of the rather thick berms at certain of the piezometer sites has practically eliminated the occurrence of sand boils at the landside toe of the levee, and lengthened the path of any potential piping channel which would have to develop before the levee would be endangered. However, as illustrated by occurrence of a large sand boil 200 ft from the levee at Stovall during the 1937 high water, a 100- or 200-ft-wide berm does not in itself insure complete safety against underseepage.

Underseepage Control Measures

Methods and their design

922. The control of underseepage and prevention of sand boils require some measure that will control erosional seepage and reduce excess pressures beneath the landside top stratum to a safe value. Measures that may be used are impervious riverside blankets, relief wells, landside berms, drainage blankets, drainage trenches, cutoffs, and sublevees. The choice of method depends upon a number of factors including the character of the foundation, cost, permanency, available right-of-way, maintenance, and disposal of seepage water. Impervious riverside blankets, relief wells, and seepage berms are recommended as the principal means of controlling seepage beneath levees along the middle and lower reaches of the Mississippi River; the other measures may be applicable in some situations.

923. Seepage control measures are considered necessary where observed or estimated values of h_o may be expected to equal or exceed h_c (approximately 0.75 z_t) at design flood stages.

924. For levees with a semipervious top stratum landward of the levee, riverside blankets or relief wells should be designed so that i at the toe of the levee does not exceed 0.5 to 0.6, where there is no seepage berm present. Where a landside berm with a width exceeding 100 to 150 ft is present, and additional control measures are considered necessary, riverside blankets or relief wells may be designed so that at the toe of the berm i_{max} = 0.6 to 0.7. Seepage berms should have a thickness and width, if practicable, such that i through the top stratum and berm at the landside levee toe will not exceed 0.5, and i at the berm toe will not exceed 0.75 to 0.8.

925. Where there is no natural top stratum landward of the levee, and the creep ratio is too low, riverside blankets and relief wells should be designed to intercept sufficient seepage so that the uncontrolled seepage emerging landward of the levee will be no more than about 150 to 200 gpm at a project flood stage. If seepage berms are to be constructed, the berm should be long enough to increase the creep ratio to an acceptable value, and i through the berm at the toe of the levee

should be equal to or less than 0.5.

926. Impervious riverside blankets. Where the pervious substratum is or is nearly exposed riverward of a levee, an impervious blanket, at least 3 to 5 ft thick, can be used to reduce the intensity of seepage and pressure landward. If a blanket is to be constructed in an existing borrow pit, the pit should be drained to permit the growth of willows which will reduce scour during floods. If the blanket will be subject to scour, it should be protected by means of abatis or spur dikes strategically located.

927. Relief wells. Relief wells with proper spacing and penetration will reduce substratum pressures and control seepage for almost any combination of riverside conditions, pervious foundation, and landward top strata. The wells should penetrate the principal pervious stratum to obtain efficient relief of pressure, especially where the foundation is stratified. Wells must offer little resistance to water flowing through the screen and out of the well; they must be constructed so as to prevent infiltration of sand into them after initial pumping, and to resist the deteriorative action of soil and water.

928. Disadvantages of relief wells are that they require periodic inspection and maintenance, must be protected from backflooding, and they increase the total quantity of seepage about 20 to 40 per cent depending on conditions. However, these disadvantages can be partially overcome by providing the wells with suitable guards, check valves, and standpipes to prevent flow during low flood stages.

929. Landside seepage berms. A landside berm controls seepage by increasing the thickness of the landward top stratum so that the weight of the berm and top stratum is sufficient to resist uplift pressures. A berm also lengthens the path of seepage flow, thereby reducing the tendency to failure by piping. The berm should be wide enough so that the head at the berm toe is no longer critical. A berm also affords some protection against landside sloughing of the levee as a result of seepage. Berms can be used to control seepage efficiently where the landside top stratum is relatively thin and uniform, or where no top stratum is present, but they are not efficient where the top stratum is relatively thick and high uplift pressures develop, as the thickness and

width of berm required to reduce upward seepage gradients to those recommended herein may be excessive. Berms may vary in type from impervious to completely free draining. The selection of the type berm to use should be based on availability of borrow materials and relative cost of each type.

a. Impervious berms. An impervious seepage berm restricts the natural relief of pressure that would result from seepage through the top stratum, and thus increases the hydrostatic head at the levee toe with respect to the original ground surface. The effect of an impervious berm on substratum pressures is the same as increasing the base width of the levee an amount equal to the width of the berm.

b. Semipervious berms. A semipervious berm is one in which the vertical permeability is equal to that of the top stratum. Natural seepage will pass through this type of berm if it is not too thick. On the basis of values of k_{bL} determined at the piezometer sites, it appears that for a berm to be classified as semipervious it must be constructed of silty sand or fine sand.

c. Sand berm. A sand berm should have a vertical permeability of at least 100×10^{-4} cm per sec. Even with this permeability, seepage into the berm must emerge at its surface as sand berms do not have sufficient capacity to conduct any appreciable flow landward.

d. Free-draining berm. A free-draining berm is one where the seepage enters the berm, is collected and discharged landward with low internal head losses in the berm. Such a berm will not affect the natural seepage flow pattern or distribution of landside substratum pressures and, therefore, will result in the narrowest and thinnest of all berms required for a given foundation condition. However, a free-draining berm must be underlain by properly designed sand and gravel filters and a collector system. As for relief wells, the ends of outfall pipes from the collector system should be provided with suitable check valves and outlet guards.

930. Drainage blankets. A drainage blanket can be used to control underseepage where the levee is built on exposed sand of fairly homogeneous character. However, it is not effective for controlling seepage from deep substrata where impervious strata or even stratified fine sands exist between the drain and the deeper more pervious sands. Drainage blankets are not generally considered suitable for control of underseepage along levees in the Mississippi River Valley because of the usual presence of

an upper top stratum of clays, silts, or fine sands overlying the deeper and much more pervious aquifer.

931. Drainage trenches. Drainage trenches can be used to control underseepage where the top stratum is thin and the pervious foundation is of limited depth so that the trench can be built to substantially penetrate the formation. Where the pervious foundation is deep, a drainage trench of any reasonable depth would attract only a small portion of underseepage, its effect would be local, and detrimental underseepage would bypass the trench. Because of the depth of the pervious substratum along Mississippi River levees, drainage trenches are not considered feasible for these levees. As in the case of drainage blankets, the existence of moderately impervious strata or even stratified fine sands between the bottom of the drainage trench and the underlying main sand aquifer will render ineffective or decrease the efficiency of a drainage trench.

932. Cutoffs. Where practicable, the most positive method of underseepage control is to cut off all seepage beneath a levee by means of an impervious barrier which will eliminate both excess substratum pressures and the problem of seepage water landward. However, completely cutting off pervious strata 80 to 200 ft deep along extensive reaches of levees is not economically feasible. The installation of partially penetrating cutoffs will not reduce seepage and excess pressures significantly unless the cutoff penetrates 95 per cent or more of the pervious aquifer. However, shallow cutoffs along the riverside toe of the levee are feasible where it is desired to cut off relatively thin layers of either natural levee or crevasse sands lying immediately beneath the base of the levee and which are in turn underlain by more impervious strata.

933. Sublevees. A landside sublevee can be used to control seepage by storing water over an area to provide a counterweight against excess head beneath the top stratum in the subleveed area. Sublevees can be used to control seepage where the landside top stratum is relatively thin, and in low areas where seepage normally ponds. Sublevees have the disadvantage that if sand boils occur within the subleveed area they may be difficult to detect or observe and may not readily be given emergency treatment, if needed. Control of seepage by sublevees requires proper

manipulation of water levels in the sublevee basins during a high water. Controlling underseepage by means of substandard sublevees is potentially hazardous as failure of a sublevee when full of water would result in losing the counterhead at a critical time.

Construction and maintenance of control measures

934. Relief wells. Relief wells can be installed in a hole made by either the reverse rotary method, the casing method, or other method that will not remove excess material from the foundation. The reverse rotary method of drilling well holes in sand is basically drilling by suction since material is removed by suction pipe. The walls of the hole are supported by seepage forces acting against a thin film of fine-grained soil on the walls, created by maintaining a head of water in the hole several feet above the ground-water table. A temporary casing may be used to support the walls of a hole during drilling and placing of well screen, riser pipe, and gravel filter. It may be set by any approved method that will not create a cavity around the outside of the casing.

935. A typical relief well consists of a wooden screen section, riser pipe, gravel filter, sand backfill from the top of the filter to within 10 ft of the well outlet, and concrete backfill from the top of sand backfill to the ground surface. Wooden screens for relief wells are usually perforated with 3/16-in. slots with a total slot area for an 8-in. ID screen of about 30 sq in. per linear foot of screen. The top of the well screen should be set about 4 ft below the top of medium to fine sand; screen should not be set in strata of clay, silt, silty sand, or very fine sand; and the depth of well hole should be at least 4 ft deeper than the bottom of the screen.

936. Filter gravel for relief wells should be placed by tremie to prevent segregation of gravel as it is placed. Material for the filter should be of washed gravel, should comply with design specifications, and should not be skip-graded.

937. A well should be properly developed after installation by surging and pumping to remove the muddy water used in advancing the hole and to develop the filter. After the well has been developed it should be

subjected to a pumping test during which the rate of flow, drawdown, and the rate of sand infiltration into the well should be carefully determined.

938. The tops of relief wells should be protected with a suitable guard, and backflow of surface water into the well should be prevented by an effective rubber gasket and check valve.

939. Relief well flow is somewhat greater than natural seepage during relatively low flood stages. This increase in seepage can be minimized by p. iding each well with a standpipe to raise its discharge elevation 1 to 3 ft above natural ground surface. As soon as artesian pressure develops to such extent that water begins to spill over the top of the standpipe, it should be removed and the well system allowed to operate as originally designed.

940. In order to insure continued and proper functioning, relief wells must be kept free of sand, silt, organic matter, or any other material that would retard free flow or clog the filter, and should be inspected once a year, preferably immediately prior to normal high-water seasons, for detection of abuse by carelessness or vandalism.

941. All wells should be pumped at least every 5 to 8 years to clean them and as a check on their performance. Individual wells known to have been subjected to inflow of muddy water resulting from inoperative check valves or removal of check valves, etc., should be pumped and/or cleaned before the next high-water season. Check valves and gaskets should be checked and cleaned so that they operate properly. Collector ditches or pipes for relief wells should also be properly maintained. Mowing and weed spraying should be extended at least 5 ft beyond each well. Normal well operation requires removal of standpipes as soon as overflow of the standpipes is noted. Check valves and gaskets should be left on the well at all times. Flow from selected wells in reaches of well systems should be measured periodically to check on design assumptions and to provide a record of the amount of water being discharged.

942. Piezometers. Piezometers should be used to check the performance of seepage control measures. They should be installed in a hole advanced with an auger or bailer and casing. Samples should be taken at

every change in type of soil or at intervals not to exceed 3 ft in order to obtain representative samples of the various strata encountered. A good type of piezometer is one with a brass or bronze screen, a galvanized steel riser pipe, and plastic coupling between the screen and riser. The filter around the piezometer tip should consist of clean, well-graded medium to fine sand.

943. The piezometer should be pumped after installation until a clear stream of water is obtained. If this is not possible, the piezometer should be filled with clear water and the rate at which the water falls in the riser pipe observed and recorded as a check on its performance.

944. Piezometers should be inspected once a year prior to the usual high-water period and given any maintenance indicated.

945. Seepage berms. Seepage berms can be constructed by hydraulic fill method or by hauling. They do not require any special compaction other than that resulting from placement operations. Special precautions must be taken in construction of free-draining berms to insure that the filter layers and collector system are properly constructed. Berms should be maintained so that they will drain and be kept free of objectionable weeds and brush.

946. Riverside blankets. Impervious riverside blankets should be constructed of the most impervious material readily available, placed in 6- to 12-in. lifts and compacted with spreading and hauling equipment. Borrow for the blanket should be obtained 1000 to 1500 ft or more from the riverside levee toe, and operations should be controlled so as to leave a blanket of clay or silt at least 5 ft thick over the underlying pervious sands. Riverside blankets should be repaired if scoured, and protected by means of abatis dikes if necessary.

947. Sublevees. Sublevees constructed around critical seepage areas should have a minimum crown width of 5 ft and side slopes of 1 on 2-1/2. They should be built of relatively impervious soil and compacted. Borrow for sublevees should never be obtained between the levee and sublevee. Sublevee basins should be provided with paved overflow spillways to prevent crevassing in event of overflow as a result of seepage or surface runoff from the levee. Sublevees and basins should be kept free of objectionable

brush and control gates, spillway , and drains should be kept in operating condition.

948. Abatis dikes. Abatis dikes usually consist of a row of posts across the borrow pit supporting 2- by 4-in. stringers. Field and snow fencing are stapled to these stringers. The dike should be protected both upstream and downstream by riprap mattresses to prevent undercutting from scour.

949. Drainage blankets. Drainage blankets should be constructed of clean, properly graded sand and gravel filters. As proper functioning of a drainage blanket depends on its continued perviousness, it should not be constructed until after the landside slope of the levee has become stabilized and is covered with sod, so that soil carried by surface runoff and erosion of the levee will not clog the blanket. If the blanket must be constructed when the levee is built or before it has been sodded, an interceptor dike should be built at the intersection of the levee slope and drainage blanket to prevent surface wash from clogging the blanket. Flow intercepted by the dike should be drained off through spaced outfall channels. The interceptor dikes and outfall channels should be removed after sod has covered the levee, and the drainage blanket completed.

950. Drainage trenches. As drainage trenches are usually excavated along the landside levee toe, the side slopes of the trench should be as steep as possible without endangering the levee slope. The trenches should be constructed during the summer or fall when the water table is lowest; even then a dewatering system may be necessary to achieve the required depth for the trench. It is imperative that the filter layers be properly graded and carefully placed. The perforated collector pipe should be of corrosion-resistant material and properly installed. Vertical risers from the collector should be provided with a rubber gasket and check valve of the type used for relief wells and protected with a metal guard. Precaution should be taken to prevent the filter or collector pipe from being flooded with muddy surface water during construction. The trench may be refilled with material removed during excavation.

951. Cutoffs. Impervious cutoffs can be constructed at the riverside toe of a levee by means of open excavation or by a trenching machine and

backfilling with impervious soil. Cutoffs placed at the riverside toe should be tied into the levee by means of an impervious blanket. Open excavation for a cutoff should be made on slopes that are stable and will not endanger the stability of the levee. A cutoff can be constructed below the water table by use of a trenching machine with the trench held open by a clay slurry, and subsequently filled with pulverized clay.

Underseepage measurements and observations during flood

952. When river stages higher than 8 ft on the levee are predicted, plans should be made for reading selected piezometers at least twice a week until after the crest is past and the stage has fallen below 8 ft. Seepage conditions should be noted when the piezometers are read. Seepage should be classified as light, medium, or heavy, and the area in which it emerges should be recorded as well as the head on the levee. The location of sand boils larger than 4 in. should be recorded, together with the elevation of seepage or surface water covering the area adjacent to piezometers or sand boils.

PART XI: CONCLUSIONS

953. The following conclusions are based on the data and studies presented in this report:

- a. Sand boils and subsurface piping during high flood stages along the Mississippi River levees are the result of excess hydrostatic pressure and seepage through deep pervious strata underlying the levees. Serious piping may undermine a levee and cause it to crevasse. The severity of underseepage is dependent upon the head on the levee, source of seepage, base width of levee, perviousness of substrata, configuration of surface geology, and characteristics of the landside top stratum.
- b. There is a definite correlation between surface geology and the location and occurrence of underseepage and sand boils. Ditches and borrow pits landward of levees also have a significant effect on the location and severity of seepage.
- c. Seepage flow and hydrostatic heads landward of a levee can be estimated from seepage formulas, and/or piezometric data, and a knowledge of foundation characteristics both riverward and landward of the levee. Data obtained from the piezometer sites can be used to estimate seepage flows and hydrostatic pressures at other sites along the levee system.
- d. Piezometer readings obtained during high-water periods provide the best data for determining the source of seepage and seepage exit, and for predicting seepage and hydrostatic heads at a project flood. Such data are extremely valuable in designing seepage control measures.
- e. Removal of the natural riverward top blanket as a result of uncontrolled borrow operations has seriously aggravated the underseepage problem along Mississippi River levees. Except where a clay stratum several feet thick has been left in place, the source of seepage at the sites studied is in the riverside borrow pits.
- f. The permeability of landside top strata is related to the type of soil, its thickness, and the presence of minute fissures and holes in the top stratum. The permeability of relatively thin top strata landward of levees appears to be considerably higher than would be obtained from laboratory tests. Landward top strata were generally more pervious than riverward top strata of the same soil type and thickness.
- g. The permeability of the pervious substratum in the Lower

Mississippi River Valley is best determined from a pumping test, but can be approximated reasonably accurately from a correlation between D_{10} and k_f as developed from field pumping tests and mechanical analyses. Coefficients of permeability as determined from laboratory tests on remolded samples of sand are not considered satisfactory for seepage analyses.

h. Underseepage can be controlled by properly designed riverside blankets, relief wells, landside seepage berms, drainage blankets or trenches, cutoffs, or sublevees. Only the first three methods are considered generally applicable for Mississippi River levees. Sublevees and drainage blankets or trenches may be applicable in certain special situations. The above control measures may be used singly or in combination.

i. Control of underseepage at problem sites along most Mississippi River levees usually requires the following general ranges in dimensions of control measures, depending on the expected head on the levee and on top strata and foundation characteristics.

(1) Impervious riverside blankets (where practical):

Width of blanket = width of borrow pit, or 1000 to 1500 ft.
Thickness of blanket = 3 to 5 ft.
Permeability of blanket = 0.01 to 0.1 x 10^{-4} cm/sec.

(2) Relief well systems:

Well penetration = 50% (effective).
Well spacing = 75 to 300 ft.
Well diameter = 8-in. ID with 6-in. gravel filter.
Length of riser pipe = 20 to 40 ft.
Length of screen = 40 to 80 ft.
Depth of well = 60 to 120 ft.

(3) Landside seepage berms:

Thickness of berm at toe of levee = 3 to 10 ft.
Width of berm = 100 to 400 ft.

j. Of the three principal control measures, only riverside blankets reduce both landward substratum pressure and seepage. Relief wells reduce substratum pressure and intercept seepage but increase the total seepage approximately 20 to 40% depending on conditions. Seepage berms generally increase the substratum pressure at the levee toe somewhat but add a counterweight to the top stratum and force the point of seepage exit landward from the levee; they decrease the total seepage approximately 10 to 25%.

k. Seepage berms at the sites investigated, except at Gammon, have largely precluded the danger of sand boils at the levee toe. They are generally thicker than required but are not wide enough to prevent sand boils or the development of critical uplift pressures at the berm toe. The spacing of the relief wells at Trotters is closer than necessary for a regular well system at the site.

l. Additional seepage control measures are considered warranted at Caruthersville, Gammon, Commerce, Trotters 51, Stovall, Farrell, Upper Francis, Lower Francis, Bolivar, Eutaw, L'Argent, and Hole-in-the-Wall.

PART XII: RECOMMENDATIONS

954. The following recommendations are made as a result of the studies described previously.

- a. All of the levees in the Memphis, Vicksburg, and New Orleans Districts should be investigated with regard to underseepage, using the procedures, methods, and criteria for safety set forth in this report.
- b. New or additional control measures should be designed, constructed or installed, and properly maintained, wherever they are indicated from the above-mentioned investigations. Use of the design formulas and criteria, construction methods, and maintenance programs as set forth in this report is recommended.
- c. Geological and soil conditions have an important bearing on underseepage and should be given careful consideration in the location of new levees.
- d. The effect of borrow operations on the seepage problem and required control measures should be considered in the design and construction of a levee. Preferably, borrow operations riverward of a levee should be limited so as to insure leaving a minimum riverside blanket of silt or clay 5 ft thick. Where borrow operations leave only a relatively thin riverside blanket, the borrow pits should be drained to promote the growth of willows and, if necessary, should be protected against scour.
- e. The existing piezometer systems should be maintained and observed during significant high waters and the data analyzed as they become available. (The project flood stage at most of the sites is considerably higher than the river stages experienced since installation of the piezometers; therefore, valuable information is still to be gained from higher river stages.) The natural seepage should also be observed at or near the crest of future high waters at the sites where it was measured in 1950.

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APPENDIX A: SAND MODEL STUDIES OF RELIEF WELLS

1. Prior to the initiation of this study in 1940, most of the theoretical and model studies made in connection with the design of relief well systems had been made for the case of a homogeneous pervious foundation overlain by an impervious top stratum with a vertical seepage entrance. A number of sand models were constructed as part of the general underseepage investigation to study the phenomena of underseepage and its control by means of relief wells. Some results of these experiments were included in Part VI of this report, and certain other graphs and summaries of test results from two of the models described in reference 43 have been selected for presentation in this appendix for the purpose of illustrating certain principles of underseepage and the action of relief wells.

Purposes of Model Studies

2. Specific purposes of the studies in the sand models were:

a. To obtain information on well flows and landward pressure for different well spacings and penetrations, and for various foundations and seepage entrances.

b. To determine the capacity of a line of wells to intercept underseepage where the landside top stratum is not impervious.

c. To determine the increase of total flow caused by wells where there is natural seepage without wells.

d. To verify, where applicable, available design data obtained from theoretical and electrical model studies for homogeneous foundation conditions.

e. To study the effect of stratification on the efficiency of a well system as regards well penetration.

Description of Models and Tests

3. The models for which certain data are presented in this appendix are illustrated in figs. A1 and A2. Model A had a homogeneous

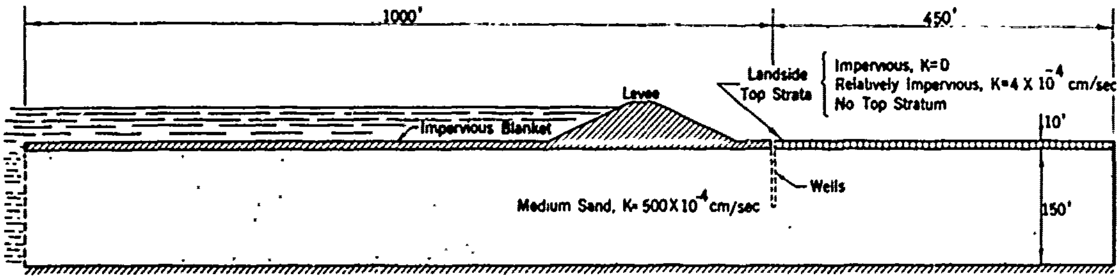


Fig. A1. Model A-a. Homogeneous foundation model

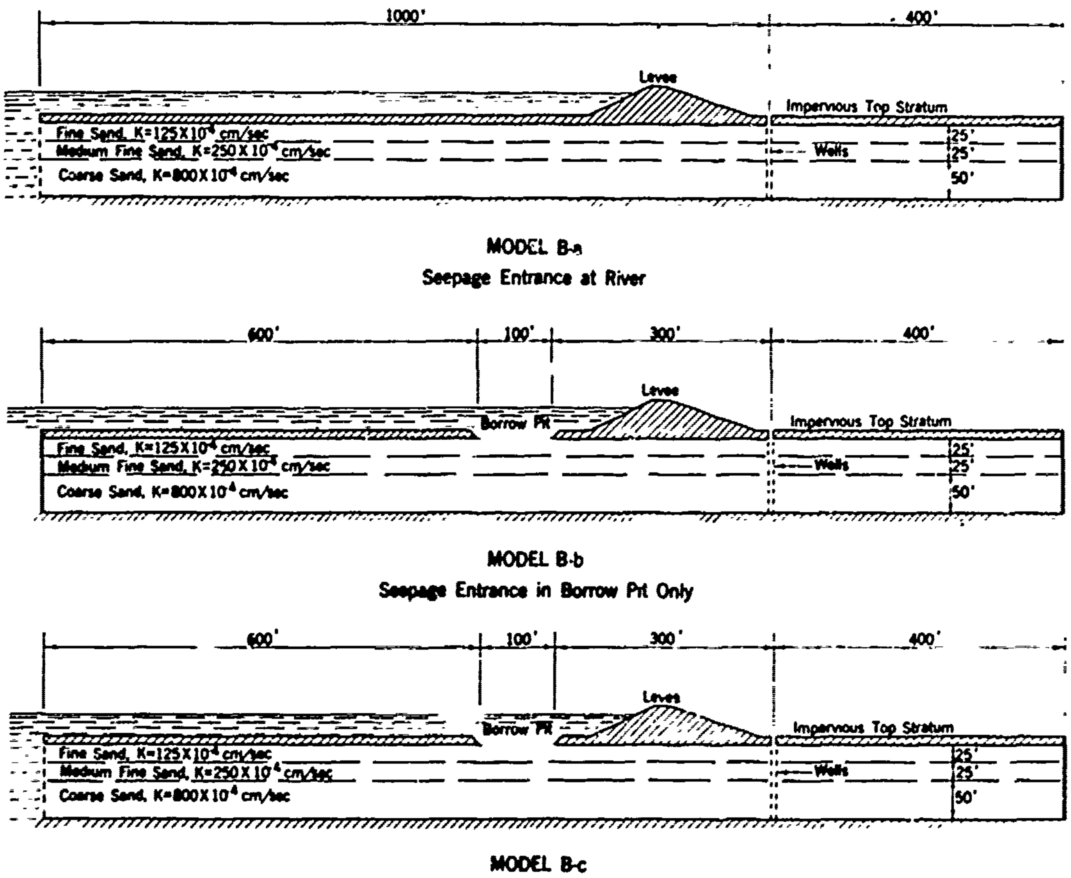


Fig. A2. Model B. Stratified foundation model

foundation of sand with various types of landward top strata. Model B had an impervious landward top stratum and a stratified foundation of sand with various seepage entrances. All dimensions, well diameters and spacings, seepage and well flows, and head measurements in the models

have been adjusted to prototype units in this appendix. Well and seepage flows are given in gpm per ft of net head, and hydrostatic pressures are given as a percentage of the total net head. Other information regarding construction and testing the models may be found in reference 43.

Test Results

4. The results of certain selected tests in models A and B are shown in the following figures. All data regarding seepage entrance, foundation conditions, landside top strata, well spacing, and penetration are presented in each figure. The data shown pertain only to the particular model and well systems described. Application of the model results to any specific field problem would require that proper consideration be given to any change in well diameter and penetration, foundation conditions, seepage entrance, or top strata, from that tested in the model. The term "seepage" as used in the figures for model A is limited to the seepage or water rising to the surface through the top stratum landward of the line of relief wells.

Model A-a

5. Well flow and the head midway between wells at the landside toe of the levee as observed for three different landside top strata in model A-a for various well penetrations and spacings are shown in figs. A3 and A4. The effect of the three types of landside top stratum on well flow and head between wells was shown in fig. 55.

6. Relief wells not only may be used to reduce substratum pressures but also to intercept underseepage with an attendant reduction of the natural seepage that otherwise would rise to the surface through the top stratum landward of a line of relief wells. Interception of seepage and the relationship between well flow and natural seepage are shown for a leaking top stratum (model A-a-2) in fig. 56 and for no landside top stratum (model A-a-3) in fig. A5. The increase of flow due to wells in excess of that naturally occurring without wells is also shown on these figures

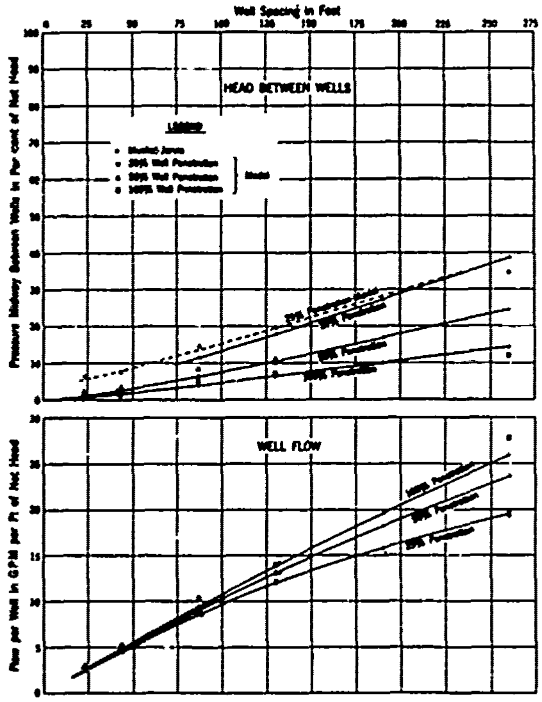


Fig. A3. Model A-a-1. Impervious landside top stratum. Well flows and landside substratum pressures

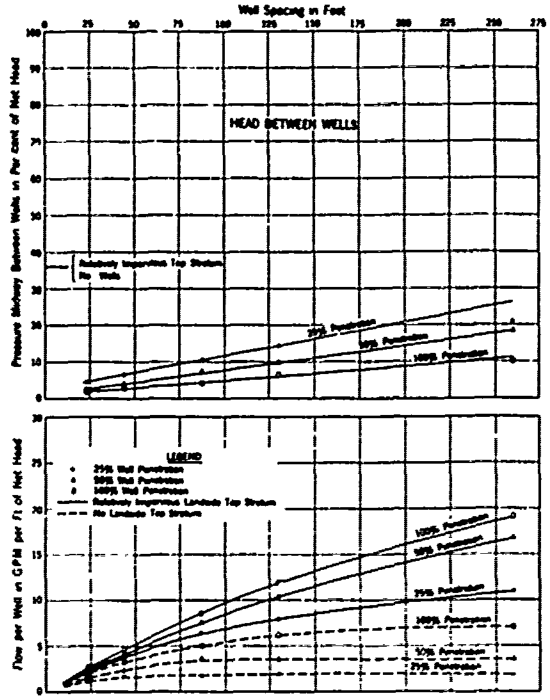


Fig. A4. Models A-a-2 and A-a-3. Relatively impervious and no landside top stratum. Well flows and landside substratum pressures

Model B

7. Well flow and the maximum head landward of the wells as observed for three different seepage entrance conditions tested in model B for various well penetrations and spacings are shown in figs. A6-A8.

8. Hydrostatic pressures, as measured by piezometers beneath the top stratum, are shown for models B-a, B-b, and B-c in fig. 57. Equipotential lines drawn from piezometric data and flow lines obtained from dye lines are shown for these models in fig. 58.

9. The test data presented in figs. A6-A8 show that in order to achieve effective pressure relief in stratified foundations, the well screens must penetrate into the principal seepage-carrying strata. It may be noted that the 100-ft-wide open borrow pit in model B-b permitted more water (approximately 25%) to enter the pervious foundation than entered the foundation with a vertical seepage entrance 1000 ft from the well line. The distance from the well line to the "effective" seepage

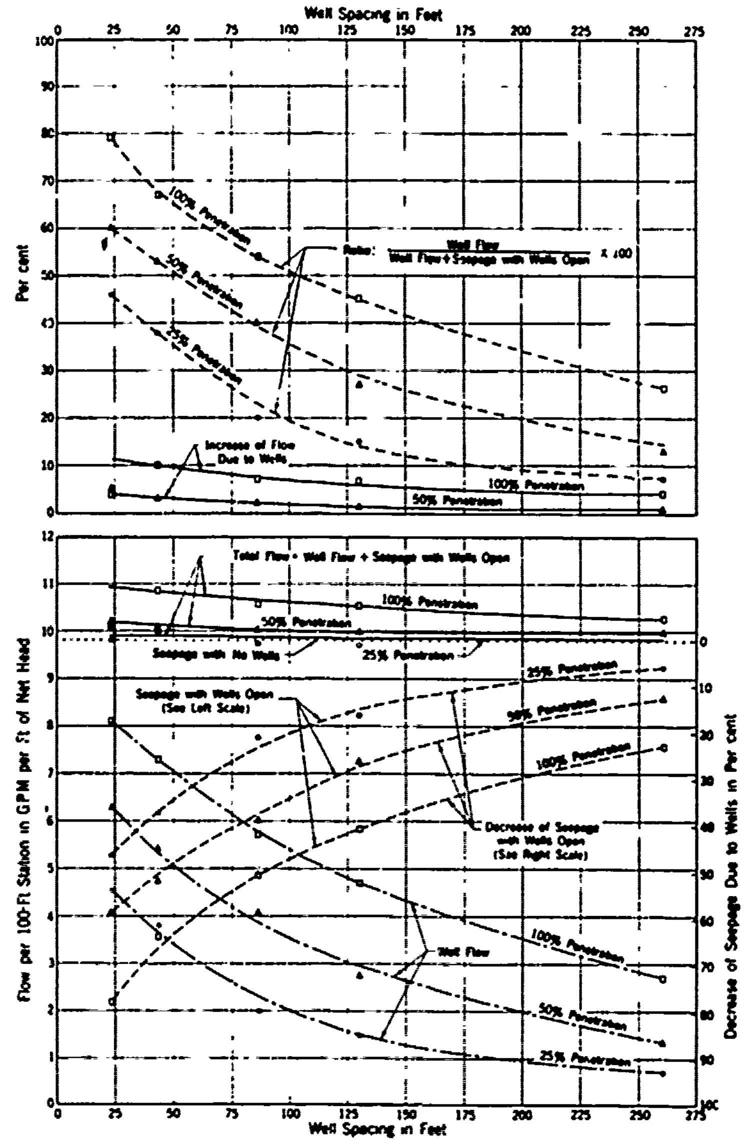


Fig. A5. Model A-a-3. No landside top stratum. Well flow and seepage

entrance in model B-b was about 725 ft as determined graphically by the method described in Part III of this report. In model B-c, the distance to the effective source of seepage entry was about 550 ft. Thus, borrow pits, excavated to sand, riverward of a levee materially reduce the distance to the source of seepage entry, and correspondingly increase seepage flow and substratum pressures so that more relief wells or longer and thicker berms are required to control underseepage.

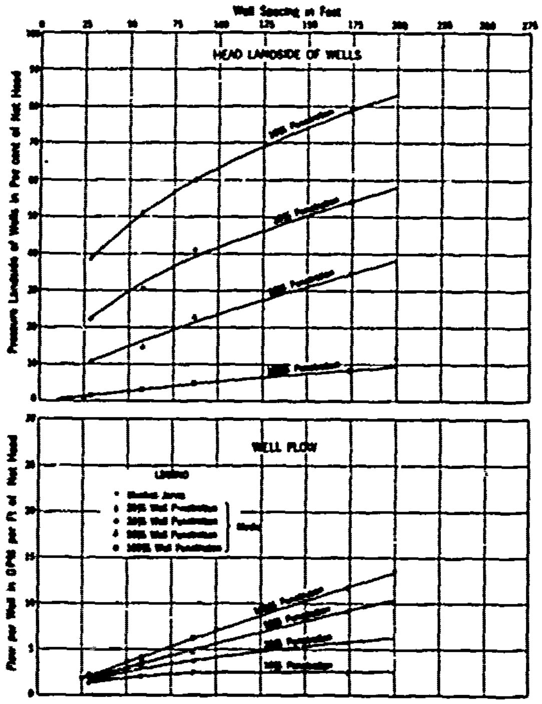


Fig. A6. Model B-a. Seepage entrance in river. Well flows and landside substratum pressures

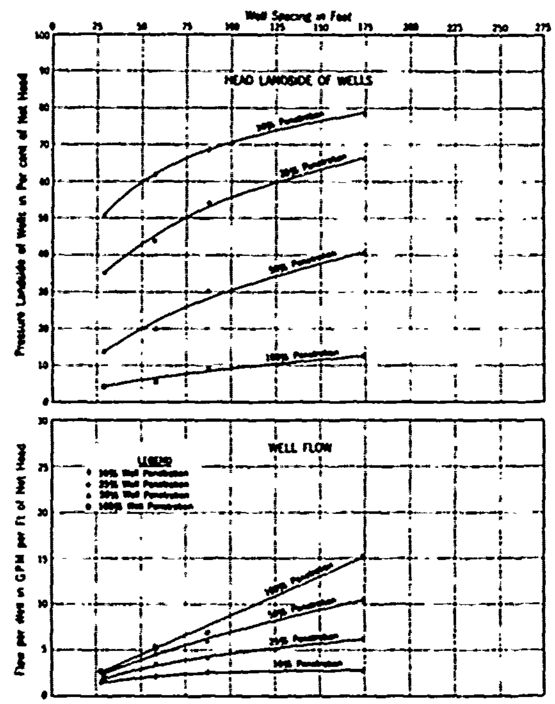


Fig. A7. Model B-b. Seepage entrance in riverside borrow pit. Well flows and landside substratum pressures

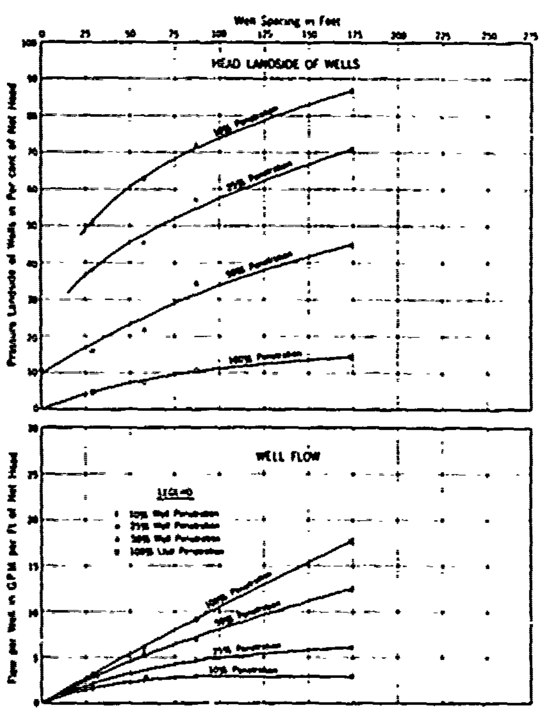


Fig. A8. Model B-c. Seepage entrance in river and in riverside borrow pit. Well flows and landside substratum pressures

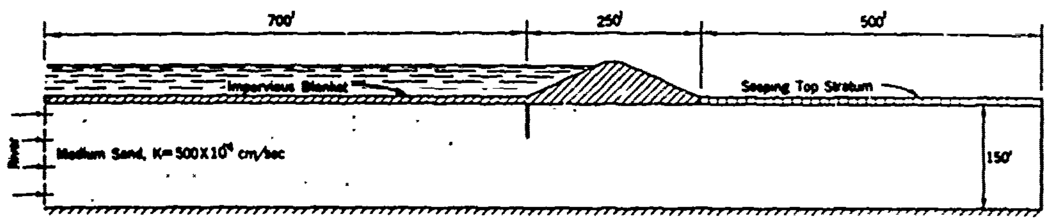
APPENDIX B: MODEL AND THEORETICAL STUDIES OF PARTIAL CUTOFFS

1. A part of the general investigation of underseepage included rather comprehensive theoretical and model studies of partial cutoffs for controlling underseepage. The effect of partial cutoffs on seepage and hydrostatic pressure beneath and landward of levees was studied for various foundation and seepage entrance conditions considered to represent, at least qualitatively, certain limiting conditions commonly encountered in the Lower Mississippi River Valley. The methods of analysis used in investigating the problem included sand and electrical models, and graphical and mathematical formulas where applicable.

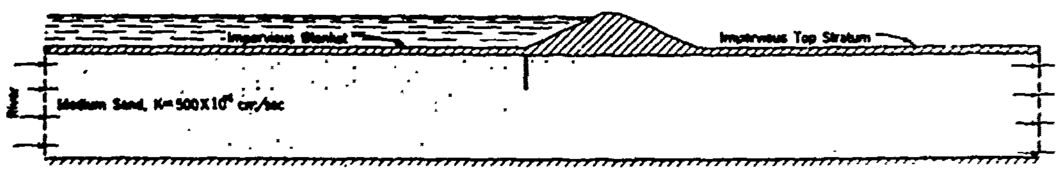
2. These studies were reported in detail in reference 40; however, some pertinent data from these studies are presented in this appendix for the purpose of illustrating certain characteristics of underseepage flow and the effect of partial cutoffs, either man-made or geological, on seepage and substratum pressures landward of levees.

Foundation and Seepage Entrance Conditions Studied

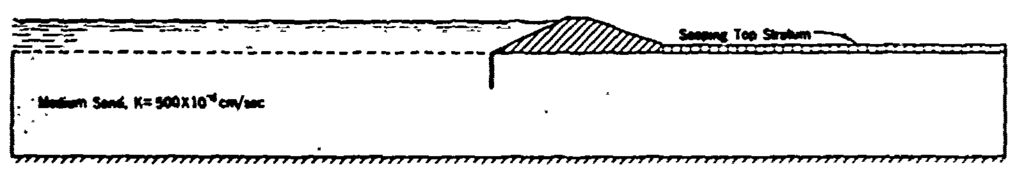
3. The basic three foundation conditions, with different seepage entrances and exits, that were studied are illustrated in figs. B1-B3 and in table B1. Results from certain selected foundation and seepage entrance conditions considered to be the most representative of conditions in the alluvial valley are presented in graphical form in the following sections. All information regarding seepage entrance and foundation conditions, landside top stratum and cutoff penetration, pertinent to the data, is presented on each figure as is the method of study. Information regarding details of the models and methods of making the tests are given in reference 40.



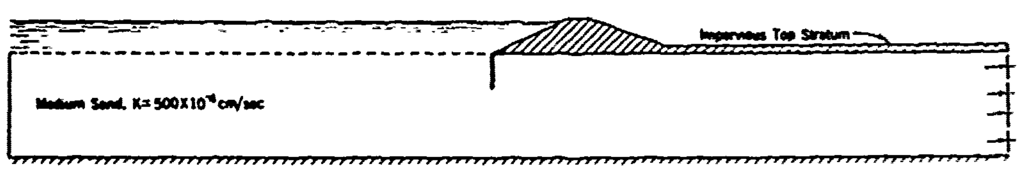
CASES I-a-1,-2,-3,-4



CASE I-a-5



CASES I-b-1,-2,-3

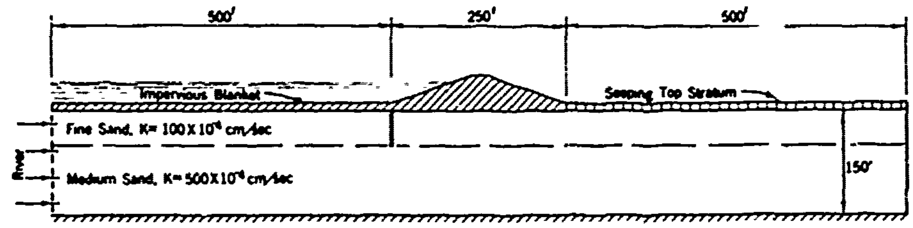


CASE I-b-4

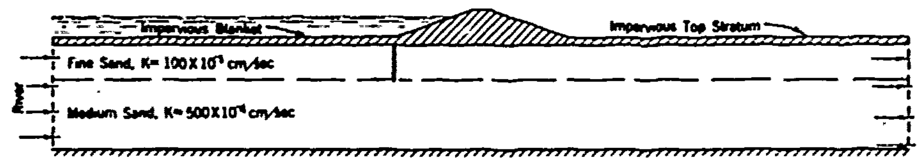
Fig. B1. Homogeneous foundation, case I

Homogeneous Foundation, Case I

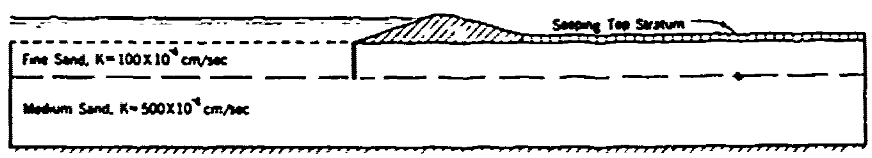
4. Mathematical formulas for determining seepage flow and heads for partial cutoffs into a homogeneous foundation are given in fig. 71. The hydraulic grade line beneath and landward of a levee with and without partial cutoffs is illustrated in fig. 72 for a leaking landside top stratum (model I-a-b). Seepage flow and head at the landside toe of a levee for various cutoffs are shown for model I-a in fig. B4. Equipotential lines and graphical flow nets, as obtained for zero and 50% cutoffs in models I-a-2 and I-a-5, are shown in fig. B5.



CASES II-a-1,-2,-3,-4

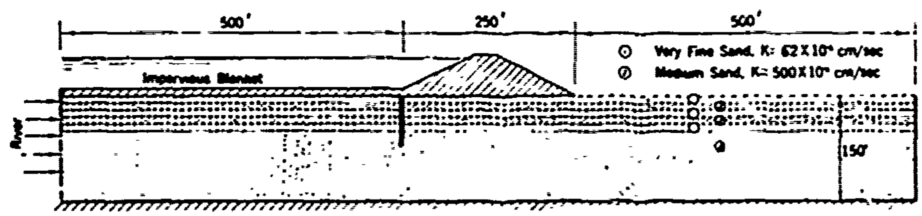


CASE II-a-5

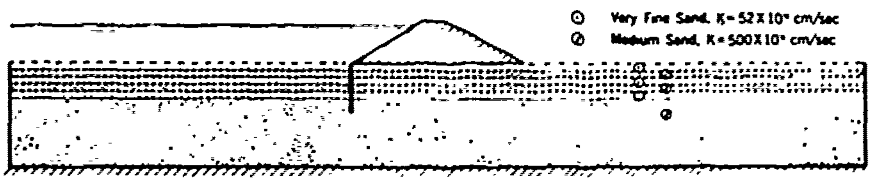


CASES II-b-1,-2,

Fig. B2. Two-layer foundation, case II



CASE III-a



CASE III-b

Fig. B3. Stratified foundation, case III

Table B1

Summary of Conditions Investigated and Methods of Analyses for Partial Cutoffs

Case*	Pervious Strata	Seepage Entrance	Seepage Exit	Landside Top Stratum	Method of Analysis
I-a-1	150 ft medium sand, $k = 500 \times 10^{-4}$ cm/sec	At river 700 ft riverside of cutoff (riverside top stratum impervious)	500-ft top stratum, 49% H at (LS) levee toe with no cutoff	A	Sand model
I-a-2			500-ft top stratum, 38% H at (LS) levee toe with no cutoff	B	Sand model
I-a-3			500-ft top stratum, 44% H at (LS) levee toe with no cutoff	C	Electrical model
I-a-4			No landside top stratum	-	Theoretical, sand model and electrical model
I-a-5			500-ft landward of (LS) levee toe (landside top stratum to this distance impervious)	-	Theoretical, graphical flow net and sand model
I-b-1**		Horizontal surface riverside of levee (no riverside top stratum)	500-ft top stratum, 68% H at (LS) levee toe with no cutoff	A	Sand model
I-b-2**			500-ft top stratum, 63% H at (LS) levee toe with no cutoff	B	Sand model
I-b-3**			No landside top stratum	-	Theoretical, graphical flow net and sand model
I-b-4**			500-ft landward of (LS) levee toe (landside top stratum to this distance impervious)	-	Theoretical and sand model
II-a-1	50 ft fine sand, ($k = 100 \times 10^{-4}$ cm/sec) underlain by 100 ft medium sand ($k = 500 \times 10^{-4}$ cm/sec)	At river 500 ft riverside of cutoff (riverside top stratum impervious)	500-ft top stratum, 75% H at (LS) levee toe with no cutoff	D	Electrical model
II-a-2			500-ft top stratum, 50% H at (LS) levee toe with no cutoff	E	Electrical model
II-a-3			500-ft top stratum, 27% H at (LS) levee toe with no cutoff	F	Electrical model
II-a-4			No landside top stratum	-	Electrical model
II-a-5			500-ft landward of (LS) levee toe (landside top stratum to this distance impervious)	-	Graphical flow net and electrical model
II-b-1**, †		500-ft horizontal surface riverside of levee (no riverside top stratum)	500-ft top stratum, 50% H at (LS) levee toe with no cutoff	G	Electrical model
II-b-2**, †			No landside top stratum	-	Graphical flow net and electrical model
III-a	10 ft very fine sand†† ($k = 62 \times 10^{-4}$ cm/sec) 10 ft medium sand ($k = 500 \times 10^{-4}$ cm/sec) 10 ft very fine sand 10 ft medium sand 10 ft very fine sand 100 ft medium sand	At river 500 ft riverside of cutoff (riverside top stratum impervious)	No landside top stratum	-	Electrical model
III-b**, †	10 ft very fine sand‡ ($k = 52 \times 10^{-4}$ cm/sec) 10 ft medium sand ($k = 500 \times 10^{-4}$ cm/sec) 10 ft very fine sand 10 ft medium sand 10 ft very fine sand 100 ft medium sand	500 ft horizontal‡‡ surface riverside of levee (no riverside top stratum)	No landside top stratum	-	Electrical model

* The words "case" and "model" are used interchangeably.

** Case was to simulate a condition similar to one sometimes created in nature when the riverside top stratum is removed during construction of the levee or dam. Entrance at river was closed for these tests.

† With a 50-ft cutoff all seepage had to pass through the upper fine sand before it could reach the deeper coarser sand.

†† Average horizontal permeability of upper 50 ft of stratified strata, $k_H = 238 \times 10^{-4}$ cm/sec; $k_V = 96 \times 10^{-4}$ cm/sec.

‡ Average horizontal permeability of upper 50 ft of stratified strata, $k_H = 231 \times 10^{-4}$ cm/sec; $k_V = 81 \times 10^{-4}$ cm/sec.

‡‡ Case was to simulate stratified conditions frequently found in the upper part of alluvial deposits.

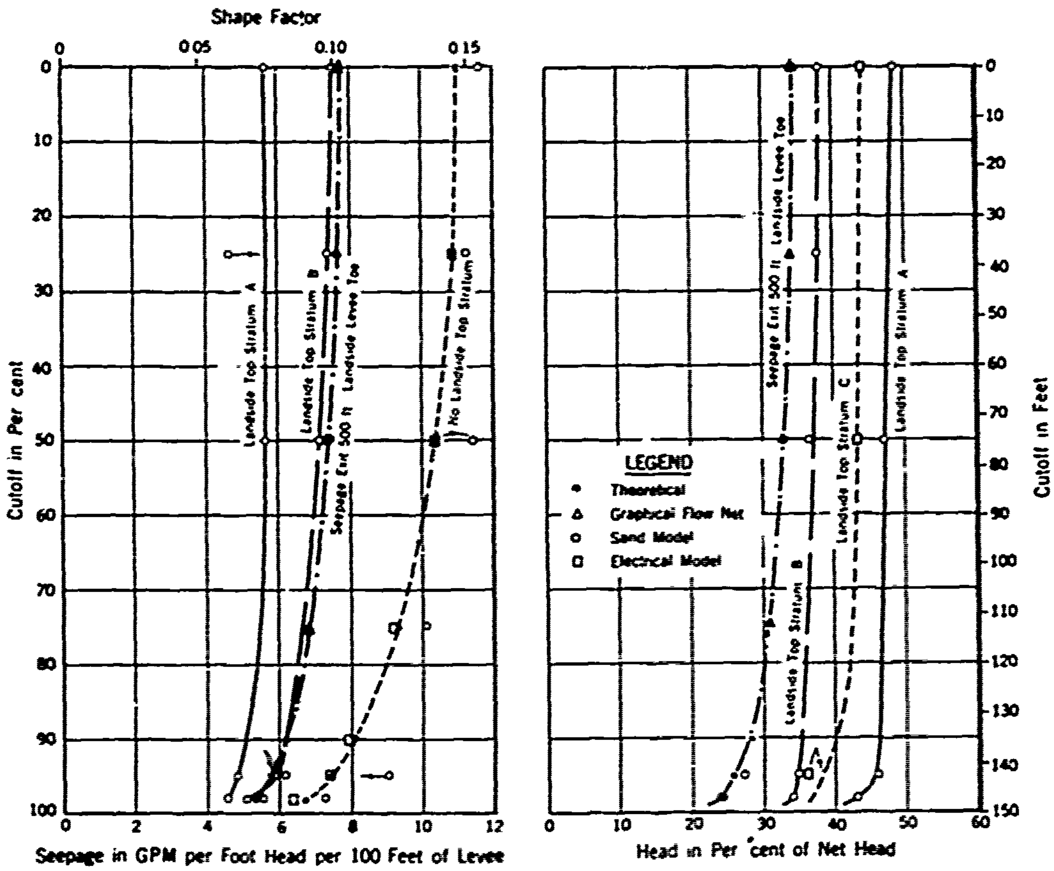
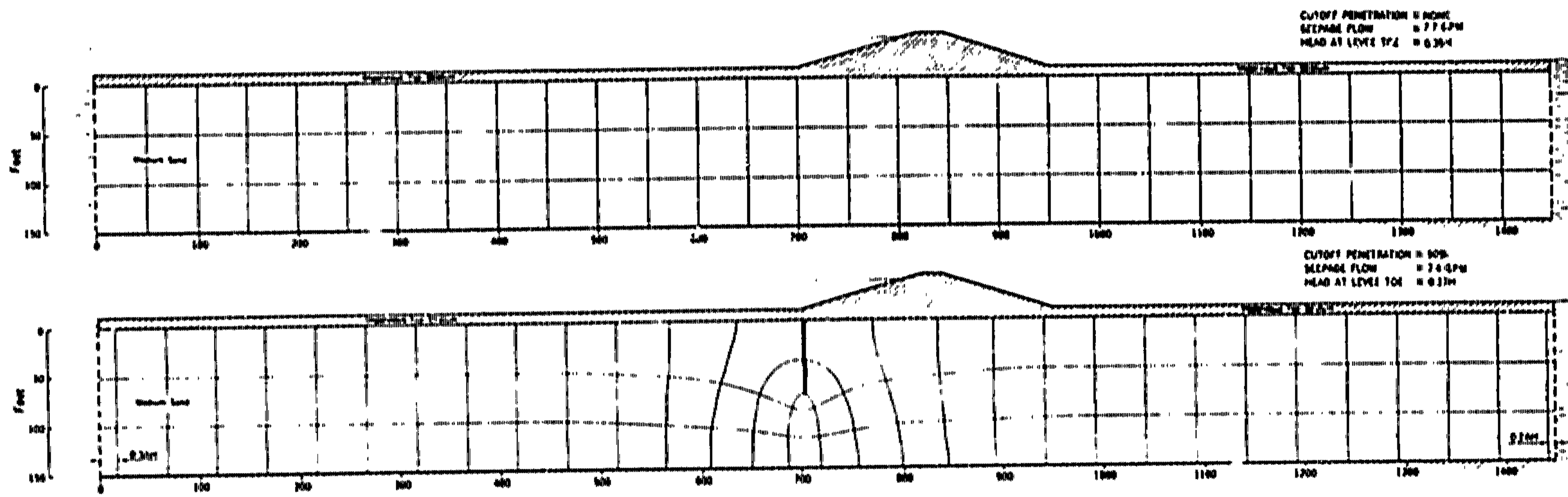


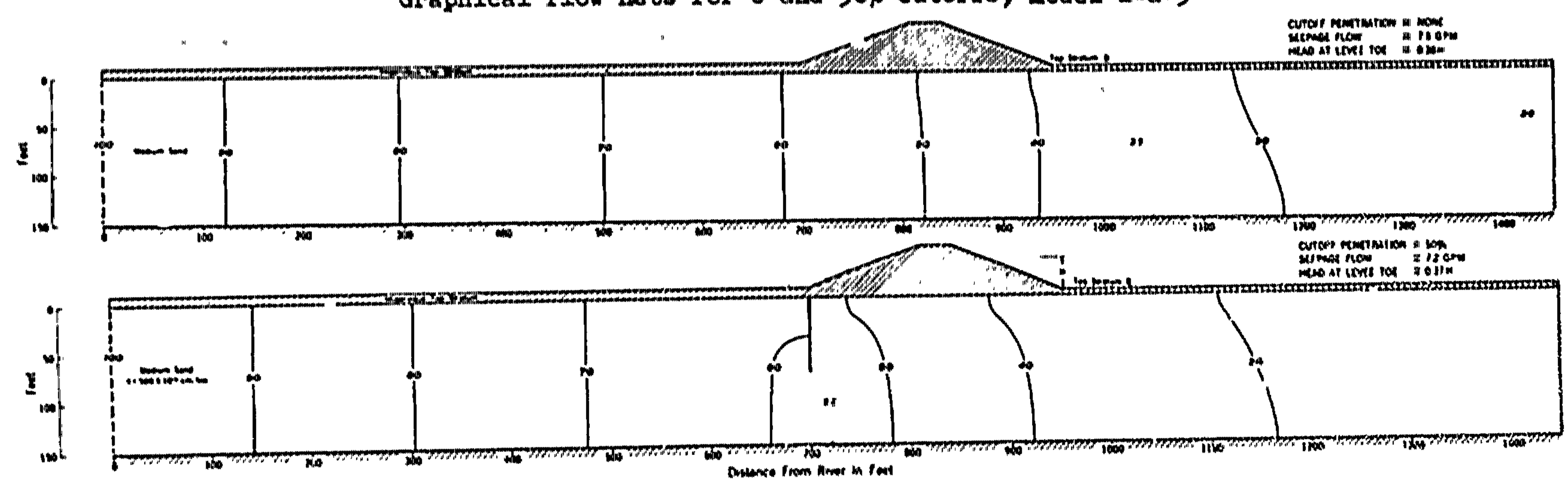
Fig. B4. Model I-a. Cutoff vs seepage flow and head at landside toe of levee. Seepage entrance at river

5. Case I-a. The results of the various methods of analyses show that partial cutoffs into a homogeneous pervious foundation have very little effect on seepage flows or landside pressures, regardless of the landside top strata. For the same seepage entrance, the more impervious the landside top stratum, the less effective was a cutoff. This is because a landside top stratum increases the resistance to flow, and the more initial resistance to flow the less effect a partial cutoff has. A cutoff of 50% reduced the seepage and landside pressure by only 1% to 5% for the landside conditions tested.

6. Case I-b. The shorter the path of seepage flow, the more effective is a partial cutoff. Therefore, partial cutoffs were slightly more effective in reducing seepage and landside pressures where the seepage entrance was at the levee toe rather than 700 ft distant as in



Graphical flow nets for 0 and 50% cutoffs, model I-a-5



Equipotential lines for 0 and 50% cutoffs, model I-a-2

Fig. B5. Model I. Equipotential lines and graphical flow nets

case I-a. However, even for this extreme entrance condition, a partial cutoff of 75% (38 ft) reduced the seepage and landside pressures only about 1% to 10%.

Two-layered Foundation, Case II

7. The hydrostatic pressure beneath the top stratum with and without cutoffs is shown for a leaking landside top stratum and two different seepage entrances (models II-a-2 and II-b-1) in fig. B5. Seepage flow and head at the landside toe for various cutoffs are shown for model II-a in fig. B6. Equipotential lines for no cutoff and a cutoff

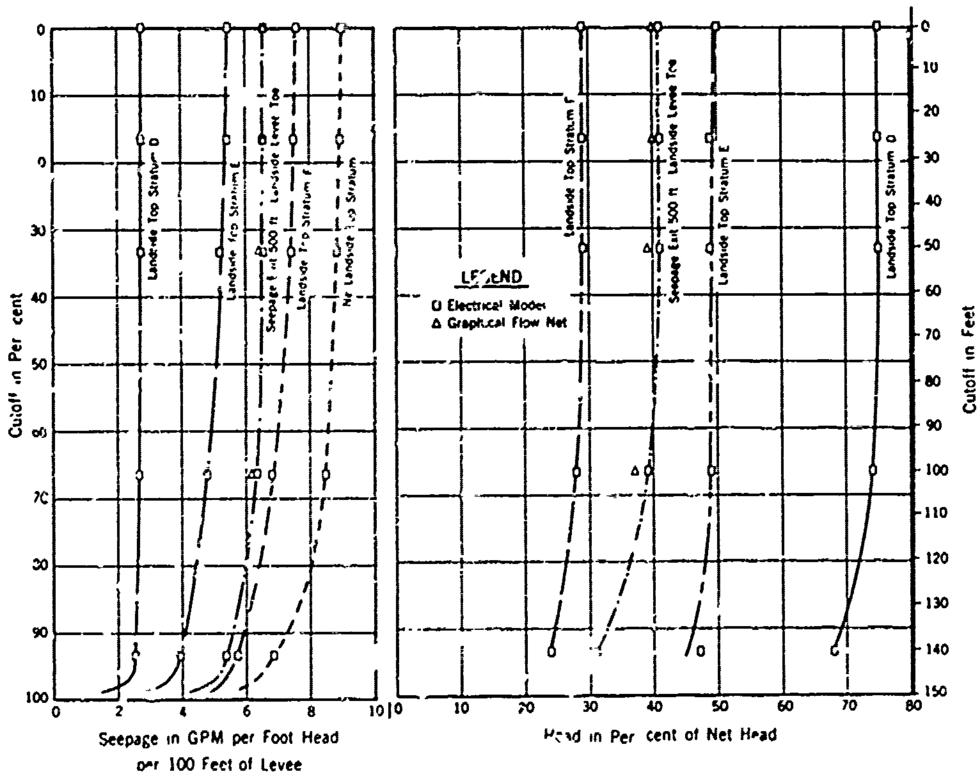
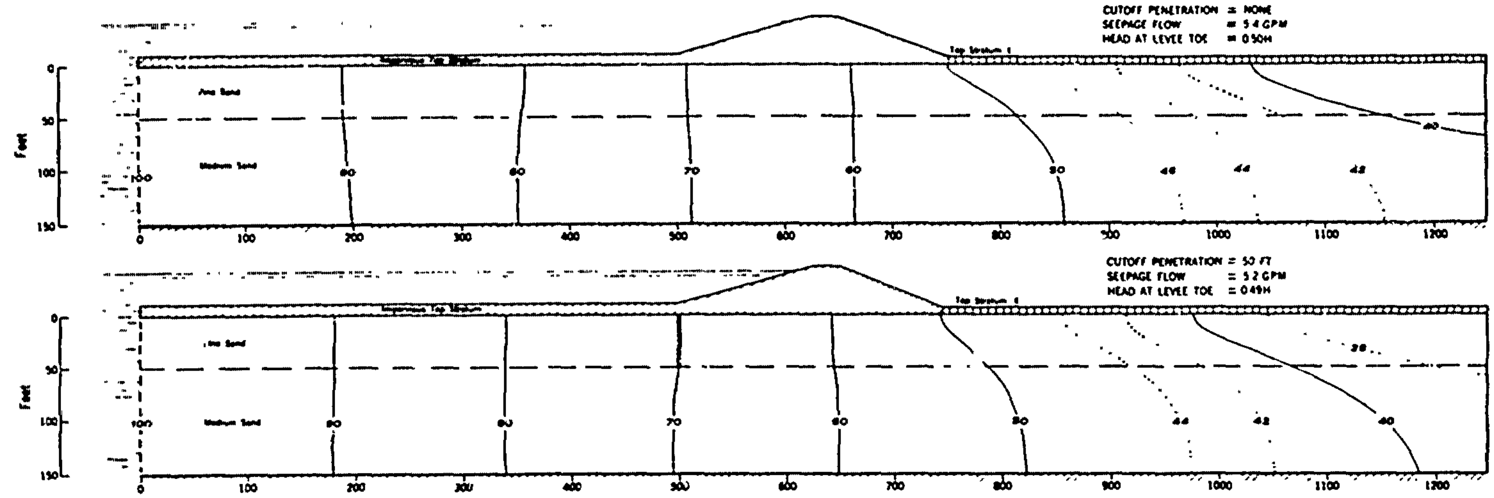
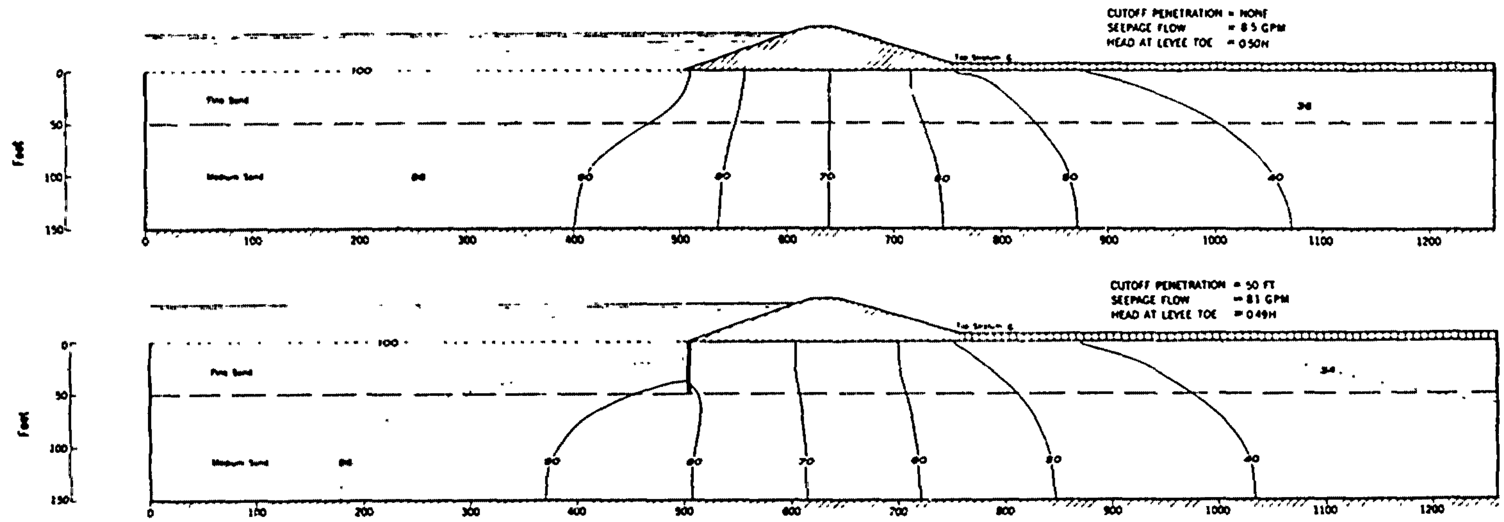


Fig. B6. Model II-a. Cutoff vs seepage flow and head at landside toe

through the upper, 50-ft, fine-sand stratum as tested in models II-a-2 and II-b-1 are shown in fig. B7. Graphical flow nets for model II-b-2 (no landside top stratum) for zero and 50-ft cutoffs are shown in fig. B8.



Model II-a-2



Model II-b-1

Fig. B7. Model II. Equipotential lines for 0- and 50-ft cutoffs

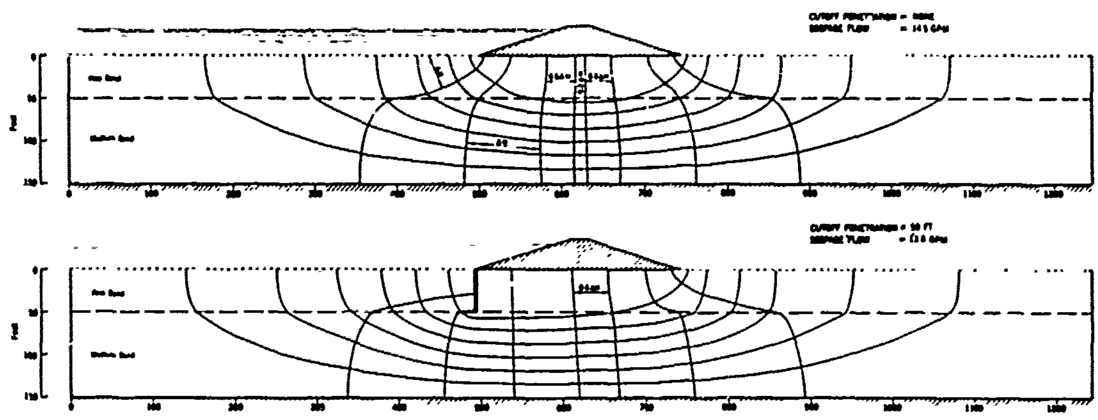


Fig. B8. Model II-b-2. Graphical flow nets for 0- and 50-ft cutoffs

8. Case II-a. In this case the deeper, more pervious sand was overlain by a 50-ft stratum of finer sand, five times less pervious. As for the homogeneous foundation, partial cutoffs had practically no effect on seepage flows or landside pressures, regardless of the landside top strata.

9. Case II-b. When the cutoff reached a depth of 50 ft in this case, all water entering the deep, more pervious sand, had to pass through the upper finer sand. As might be expected, partial cutoffs were slightly more effective in reducing seepage and landside pressures in this case than in case II-a, but again the amount of reduction was small for any moderate cutoff penetration, regardless of the landside top stratum.

Stratified Foundation, Case III

10. Seepage flow and head at the landside levee toe at the top of the deep pervious sand (depth of 50 ft) are shown for this case in fig. B9. Equipotential lines for cases II-a and II-b, with and without a cutoff extending through the upper 50 ft of stratified fine and coarse sand, are shown in fig. B10.

11. Case III-a. In this case, the deeper more pervious sand was overlain by alternate strata of very fine and medium sand, the finer sand being 3 to 10 times less pervious than the coarser sand. Where the

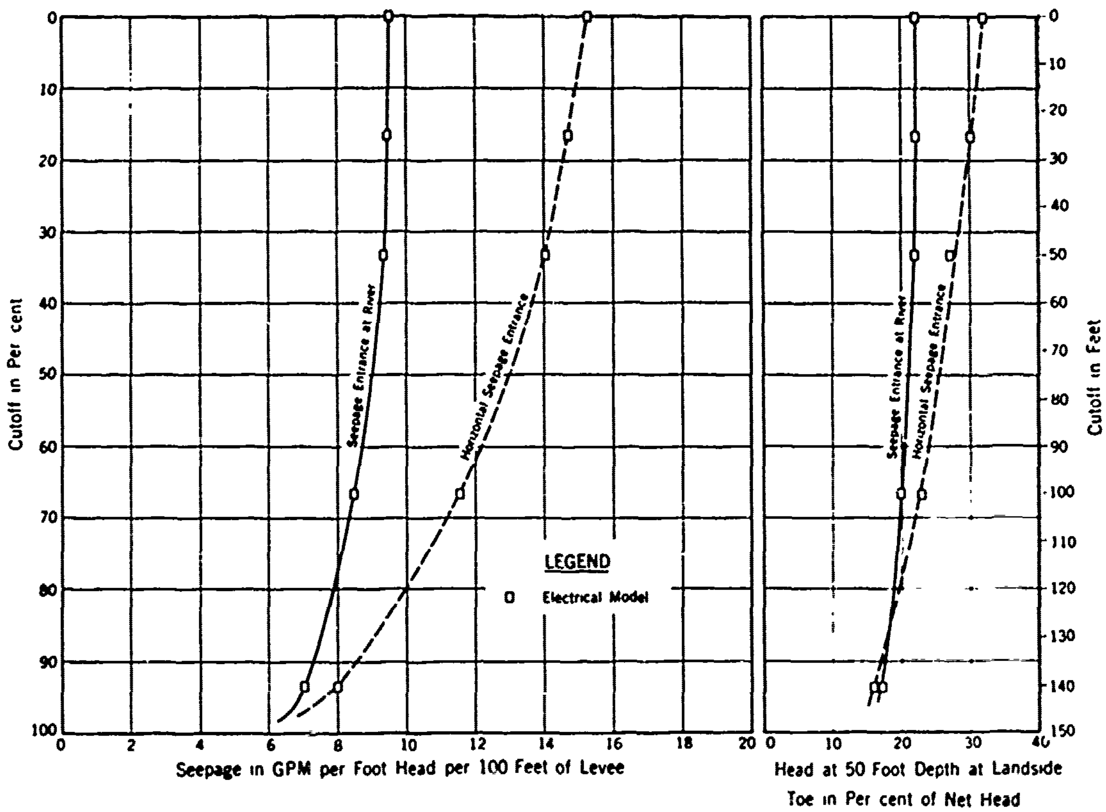
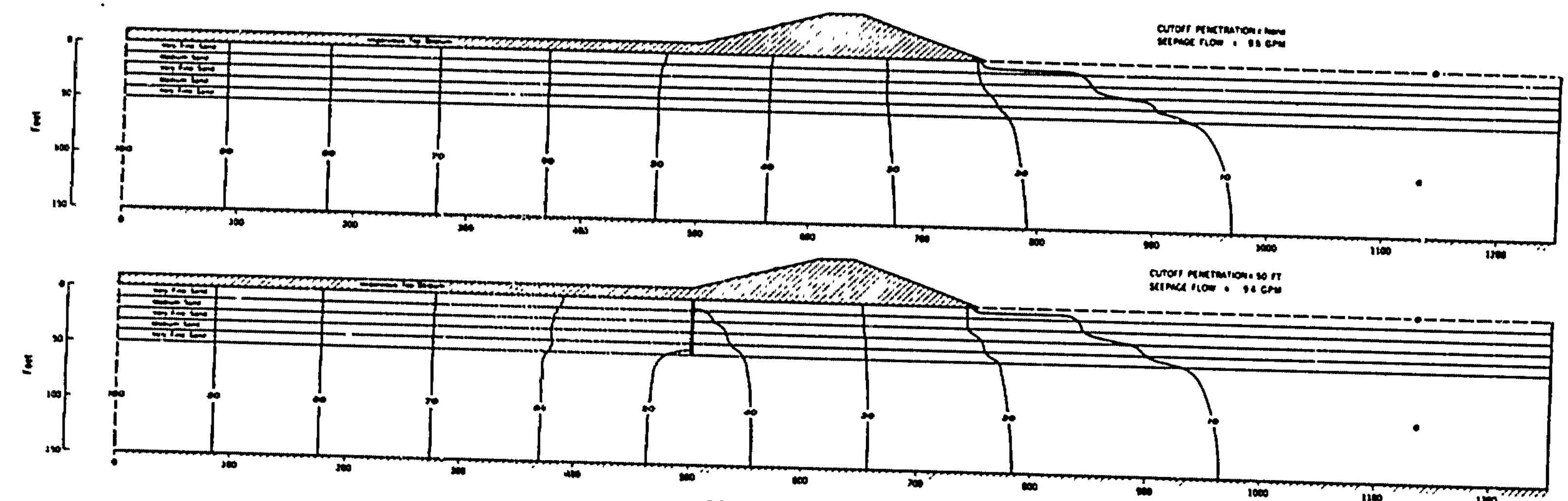


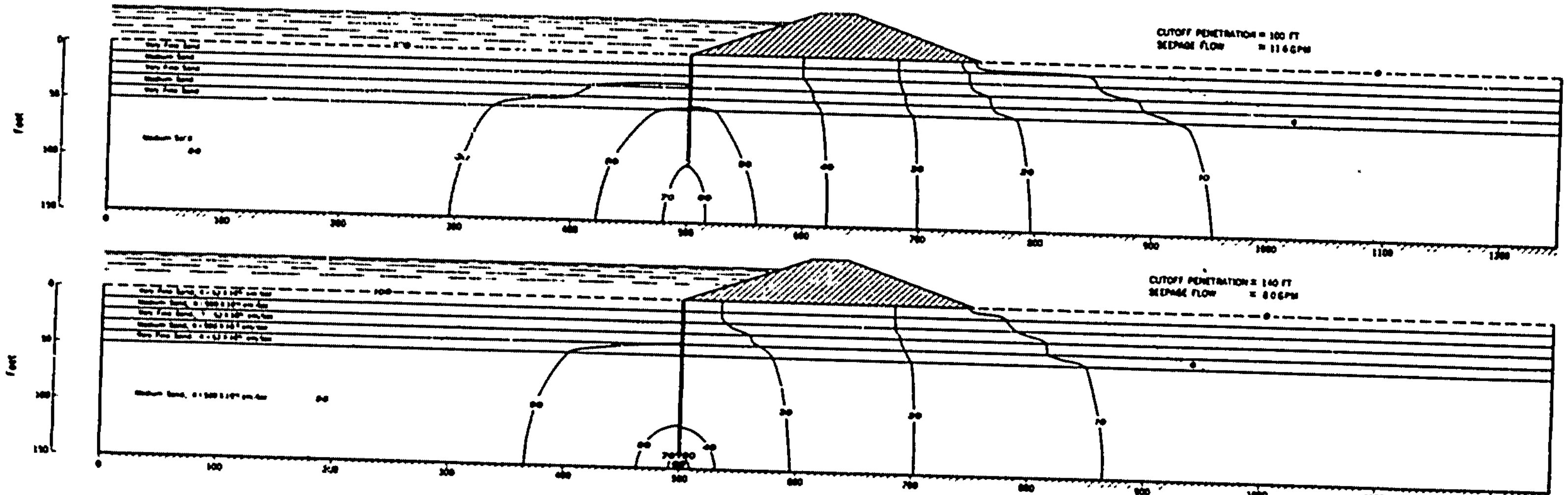
Fig. B9. Model III. Cutoff vs seepage flow and head at landside toe of levee. No landside top stratum

seepage water entered at a point 500 ft from the levee, a partial cutoff completely penetrating the stratified sand reduced the seepage flow 2% but effe[illegible] no reduction in the head at a depth of 50 ft at the landside toe of t[illegible] levee.

12. Case III-b. As in case II-b with a 50-ft cutoff, all water entering the deep, more pervious sand had to pass through the upper stratified layers of sand. Because of the entrance condition and the lack of resistance to flow (no landside blanket), partial cutoffs should be more effective in reducing underseepage and landside pressure for this case than for any other case tested. However, a 50-ft cutoff, extending through the upper stratified sands, reduced the seepage flow by only 8% and the head at the landside toe at a 50-ft depth by 18%.



Model III-a



Model III-b

Fig. B10. Model III. Equipotential lines for 0- and 50-ft cutoffs

Correlation of Methods of Analysis

13. The correlation of results obtained from the different methods of analysis is best shown on the various figures. Considering the numerous variables that enter into sand and electrical seepage models, the correlation of the results obtained from mathematical analyses, graphical flow nets, sand models, and electrical models was good. In most cases the deviation in the results was no more than 5 to 10%.

Conclusions

14. The results of the model studies and other analyses indicate the following conclusions:

- a. Partial cutoffs with penetrations less than 98% had relatively little effect in reducing underseepage or landside pressures for the range of conditions tested. Cutoffs with penetrations less than 25% had practically no effect.
- b. The longer the path of seepage flow the less effective were partial cutoffs. Correspondingly, the more impervious the landside top stratum the less effective were partial cutoffs.
- c. By similarity, it may be reasoned that not only must cutoffs completely penetrate the pervious strata, but they must also contain no openings in order to materially reduce seepage flows and landside pressure.

APPENDIX C: PERMEABILITY OF FOUNDATION SANDS

Introduction

1. One of the principal factors in the analysis of underseepage and in the design of measures to control seepage beneath levees underlain by strata of pervious sands is the coefficient of permeability of the various strata of sand. The permeability must be known for computation of seepage flow and substratum pressures landward of the levee, drawing flow nets, and determination of "effective" penetration of relief wells.

2. The importance of this factor was realized early in this investigation and numerous laboratory tests were performed to determine the grain size (D_{10}) and permeability of samples of sand taken from the pervious substratum at the piezometer sites discussed in the main report. Most of these samples of sand were remolded, having been obtained in most cases by means of bailer borings; however, a few tests were performed on undisturbed samples of sand obtained with Shelby tube samplers. The results of these tests are all shown adjacent to the logs of borings made at the piezometer sites (presented in vol. 2), and the relation between D_{10} and k_R as developed from these studies is shown on plate 244.

3. In an attempt to determine more precisely the permeability of the sand at various depths, a number of falling-head tests were made in the casing of the borings as they were advanced. However, the settling out of fines in the casing, which created a filter skin in the bottom of the hole, largely invalidated these tests and the results are not reported herein.

4. In 1943 the flow from the relief wells at Commerce and Trotters, Miss., indicated that the permeability of the sand aquifer at these sites was either considerably more than estimated from the laboratory tests, or that the source of seepage was considerably closer than assumed in the design of the systems. Therefore, it was decided to perform pumping tests to determine the permeability of the sand aquifer at Commerce and at Wilson Point, La., where a well system also had been installed. The results of these pumping tests were reported in 1949.[42] At the time

these pumping tests were performed the ground-water table before and during pumping was below the bottom of the top stratum, and for this reason it was assumed that the type of flow to the wells was of the gravity type and the data were analyzed accordingly. Since then it has been concluded that the flow to these test wells was more nearly artesian than gravity because of the relatively low permeability of the upper fine foundation sands; i.e., the flow to the wells from the principal water-carrying aquifer was bounded by material of significantly lower permeability. Because of this and for the sake of completeness, the data from the pumping test at Commerce have been reanalyzed and the results are summarized in this appendix.

5. At the time the second relief well system was installed at Trotters 54 in 1950, several pumping tests were performed on the wells to determine the over-all horizontal permeability of the sand aquifer.[45] The results of these pumping tests are summarized herein.

6. In 1953 and 1954 the horizontal permeabilities of various sand strata in situ were determined at several sites in the alluvial valley of the Mississippi River in connection with the designs of relief well systems along levees in the St. Louis District and of dewatering and seepage control facilities for structures to control Old River which links the Mississippi and Atchafalaya Rivers 45 miles south of Natchez, Miss. As the information obtained from these tests has an important bearing on the subject of permeability and design of seepage control measures, it also is included in this appendix.

Field Pumping Test, Commerce, Mississippi

7. The pumping tests at Commerce were performed on a well installed at sta 23/2+75, 325 ft landside of the levee in the line of pressure relief wells. The test well consisted of 20 ft of 5-in. riser pipe and 158 ft of screen made of 4-in. ID (6-in. OD) porous concrete. The screen and riser pipe were installed inside an 8-in. casing. No filter was placed around the porous concrete pipe. Tests were run at a well penetration of 100% and also at penetrations of about 75, 50, and

30% by progressively backfilling the well with sand. Piezometers and observation wells were spaced out from the test well at fairly close intervals on two lines, one north-south and the other east-west. Additional information regarding the well installation and pumping tests may be found in references 42 and 45.

8. The drawdown curves obtained during the pumping tests are shown in fig. C1. The curves extend from a point 10 ft from the center of the well to a distance of 1000 ft from the well. For the fully penetrating well, the drawdown curves are relatively straight lines on the semilog graph in accordance with formula 8a, and the radius of influence averaged about 1000 ft. The curvature of the drawdown curves for the tests on partially penetrating well screens in the vicinity of the well is a result of three-dimensional flow near the well. The rates at which the well was pumped and the corresponding drawdowns in the well are also shown in fig. C1.

9. The computations for the permeability of the sand substratum were based upon the upper, flat portion of the drawdown curves for all penetrations assuming the flow lines in this region of the foundation to be parallel and horizontal. In this method the permeability was computed from the formula

$$k_f = \frac{2.30\, Q_w \log_{10} \frac{r_2}{r_1}}{2\pi d\, (h_2 - h_1)} \tag{8a}$$

and the values of permeability thus computed are summarized in table C1.

10. Table C1 shows that, except for the test on the fully penetrating well where a coefficient of permeability of 615×10^{-4} cm per sec was obtained, the permeabilities from the partially penetrating well tests ranged between 885 and 1200×10^{-4} cm per sec, and averaged about 1060×10^{-4} cm per sec. The reason for the lower permeability obtained in the fully penetrating test is not known. However, since computations for the partially penetrating tests were based on the portion of the drawdown curve distant from the test well and do not involve a correction factor for partial penetration, the results should be comparable to

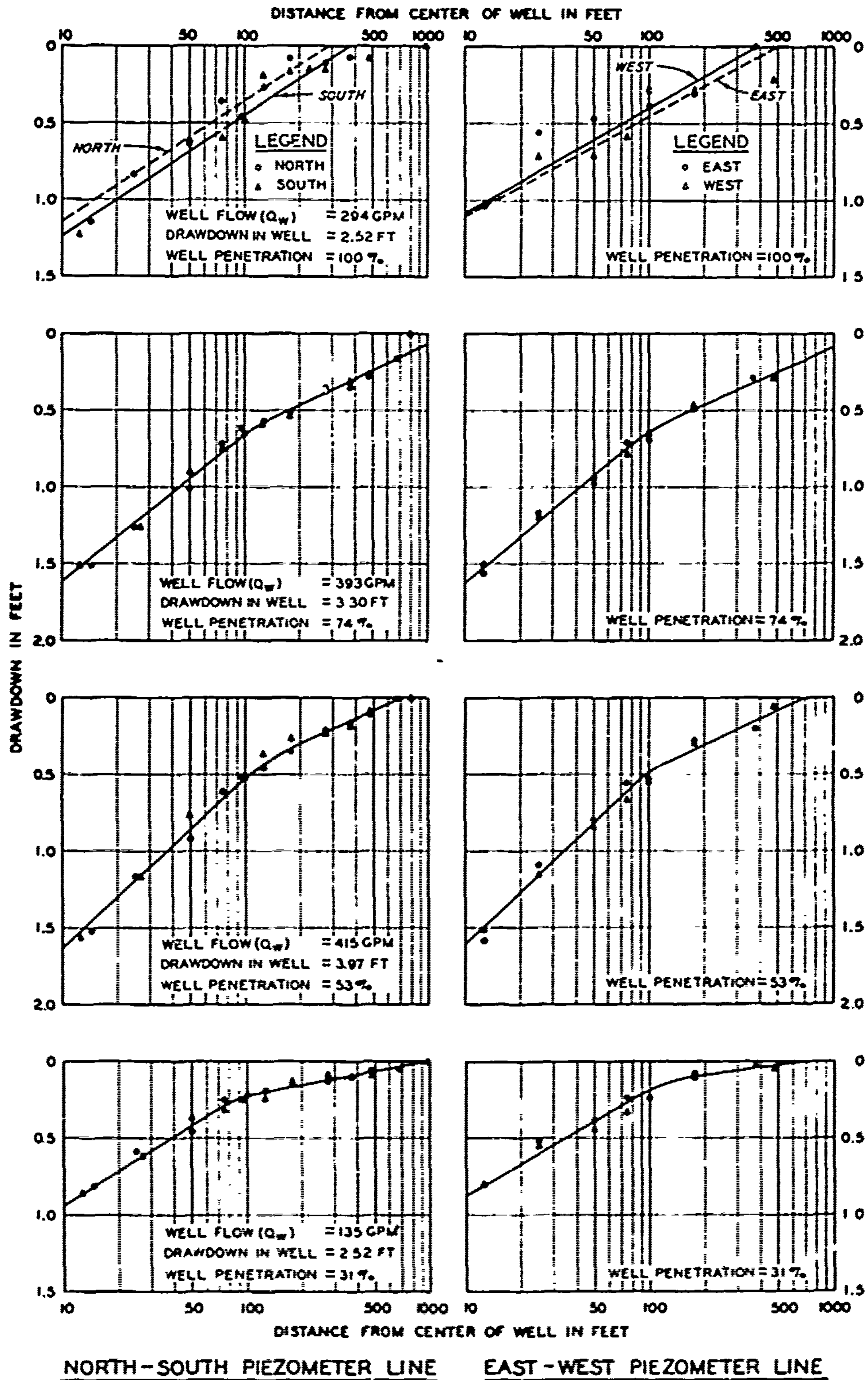


Fig. C1. Drawdown curves for pumping tests, Commerce, Miss.

test results obtained on a fully penetrating well. Therefore, it is believed that the permeability from the fully penetrating test is too low and the horizontal in-situ permeability of the substratum sands (k_f) is about 1000 x 10^{-4} cm per sec at Commerce.

Table C1

Results of Pumping Tests at Commerce

Well Penetration %	Coefficient of Permeability of Foundation Sand, 1 x 10^{-4} cm/sec N-S Line	E-W Line	Average
100	560	670	615
74	1020	1100	1060
53	1100	1060	1080
31	885	1200	1040

Average of all tests = 950

Average k_f for last three tests = 1060 x 10^{-4} cm/sec.

Field Pumping Tests, Trotters 54, Mississippi

11. During the installation of the well system at Trotters 54 in 1950, pumping tests were made on wells 1, 15, and 30 to determine the over-all horizontal permeability of the foundation sands at this site. In addition to these pumping tests for which drawdowns were observed at considerable distances from the wells, pumping tests were also performed on each well in which only the well flow and drawdown in the well were observed. Also, the head loss through the filter and well screen was measured by means of piezometers with tips placed at the outer periphery of the filter around the well screen. These latter piezometers were installed at both the top and mid-point of the well screen at wells 1, 8, 15, 22, and 30. Drawdown of the piezometric surface in the vicinity of the test wells was measured by means of piezometers installed at fairly close intervals out from the test wells.

12. The relief wells at Trotters consist of 6-in. ID redwood pipe, slotted with 3/16- to 1/4- by 2-in. slots; the slots have an open area of 18 sq in. The screen portion of the wells is 40 ft long and is surrounded with a 6-in. gravel filter. The filter has an estimated permeability of about 5000 x 10^{-4} cm/sec. The riser pipe is also of 6-in. ID redwood pipe.

13. Details regarding the well installation and the pumping tests at Trotters may be found in reference 51. Only the pumping test results pertinent to a determination of the permeability of the sand aquifer at the Trotters site are summarized in this appendix.

Drawdown curves

14. Drawdown curves obtained from the pumping tests on wells 1, 15, and 30 are shown in fig. C2. The elevation of the water in the wells for three different rates of pumping, the radius of the well screen and of the filter, elevation of the water in the piezometers at the outer periphery of the filter at the top and middle of the well screen, the water table before initiation of the pumping, and the elevation of the water in the piezometers out from the test wells, are all shown on this figure. Normally, drawdown curves for artesian flow to a fully penetrating well, when plotted on a semilog graph, are straight lines. The curvature of the lines within 100 ft of the test wells can be attributed to the fact that the well screens did not completely penetrate the pervious aquifer. The drawdown curves shown in fig. C2 indicate that the radius of influence of the wells tested ranged from about 400 to 500 ft.

Head loss through filter and screen

15. The difference in the readings of the piezometers at the top and middle of the screen for wells 1, 15, and 30 showed a significant amount of head loss in the screen and riser portion of the well up to the bottom of the suction pipe for the pump. The true specific yield for the wells was obtained by correcting the drawdown of the wells for

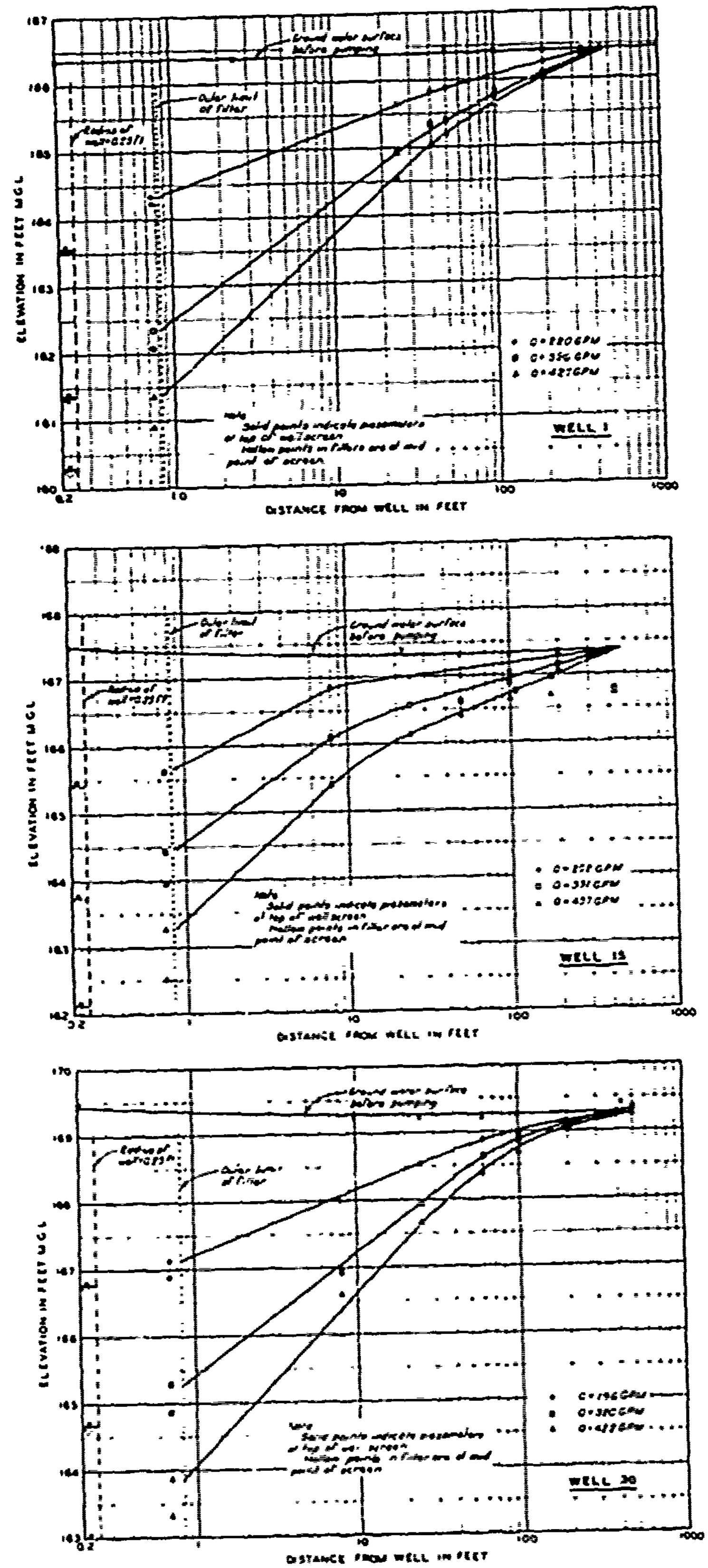


Fig. C2. Drawdown curves for pumping tests, Trotters 54

hydraulic head losses* using the data shown in fig. C3. The head loss through the filter gravel and screen at the center of the screen corresponded closely to that estimated from well tank test data. The filter

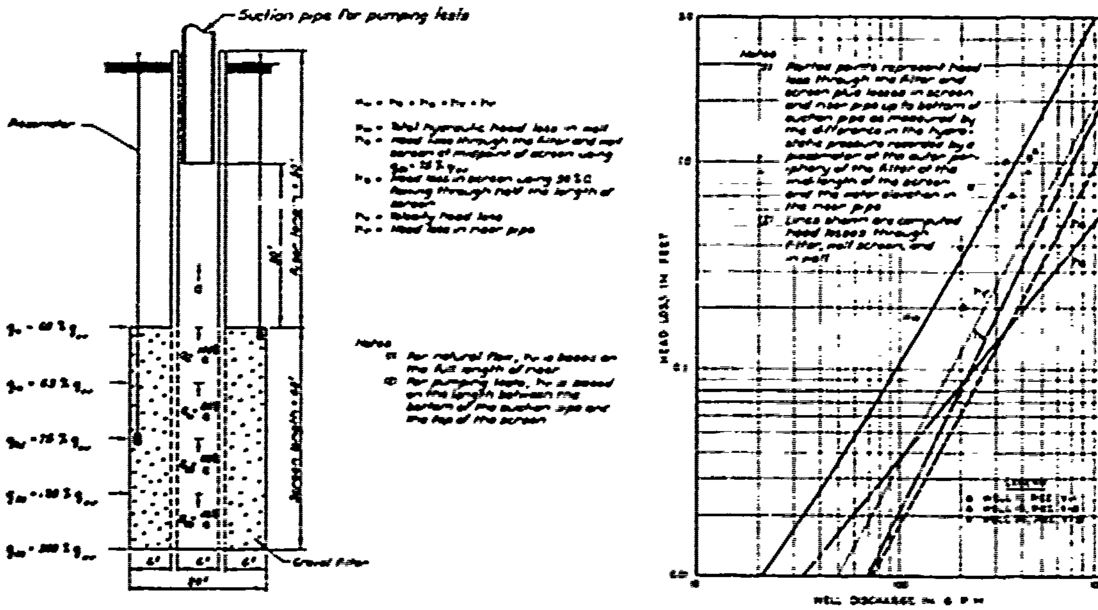


Fig. C3. Hydraulic head losses in Trotters wells

and screen loss as determined from the laboratory tests was 0.12 ft for a well flow of 5 gpm per ft of screen; the head loss through the filter and screen as measured by piezometers in the field for a flow through the screen of 5 gpm per ft ranged from about 0.05 to 0.15 ft. Thus, the filter and screen appeared to function as anticipated from the standpoint of allowing the entrance of water with a minimum of frictional resistance.

* The head loss through the filter and well screen used in correcting the drawdown in the wells was obtained from well tank test results (fig. C3) as follows: The flow measurements in the wells made during the 1951 high water, at different depths in the screen, show that the inflow at the mid-point of the screen is approximately 75% of the average screen inflow for the entire length of screen.[51] The inflow per foot at mid-point of the screen was computed for various assumed well flows and the resulting head loss obtained from fig. C4. These head losses were then plotted against the assumed well flows in fig. C3.

Specific yield of wells

16. The average specific yield for the 30 wells installed at Trotters was 100 gpm. The average specific yield for wells 6-25 was 105 gpm. The yield of wells 1-24 was extremely uniform.

Permeability of pervious substratum

17. The permeability of the pervious substratum as determined from the previously described pumping tests is summarized in table C2. The permeability of the sand stratum was computed from the slope of a secant drawn through the drawdown curves beyond 50 ft from wells 1, 15, and 30 (fig. C2) using formula 8a. In using this formula, it was assumed that the flow lines in the foundation beyond 50 ft from the well were parallel and horizontal.

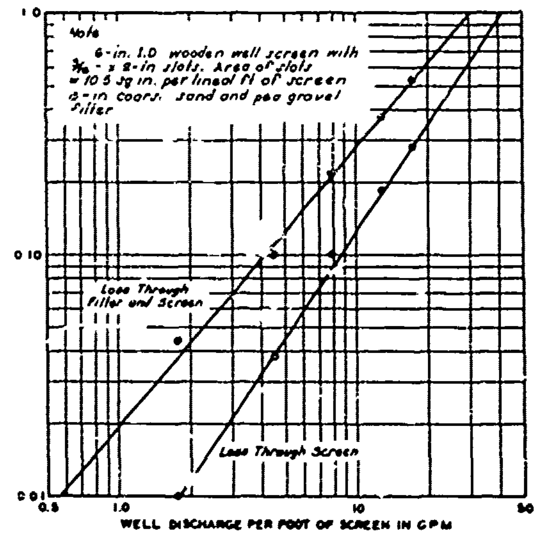


Fig. C4. Head loss through filter and well screen as measured in WES well testing tank

18. The permeability of the foundation was also computed from the drawdown in each of the wells pumped. This permeability was computed using the specific yields adjusted for head losses in well and from equation 8c for artesian flow with Muskat's correction factor of G · 0.63. The results of these computations are also summarized in table C2.

19. The average permeability of the foundation computed from the drawdown curves for wells 1, 15, and 30 was 935×10^{-4} cm per sec; the average permeability based on the specific yield of the 30 individual wells was 1180×10^{-4} cm per sec; the average permeability of the central portion of the well system based on wells 6 through 25 was 1240×10^{-4} cm per sec.

20. Well flow data and piezometric data. The permeability of the pervious stratum at Trotters was also computed from 1943 well flow and 1950 piezometer and seepage data, and 1951 well flow and piezometric data.

Table C2

Permeability of Medium to Coarse Sand Stratum

Trotters 54

	Well No.	Well Discharge gpm	$k_f \times 10^{-4}$ cm/sec
A. Permeability from Pumping Tests on Wells 1, 15, and 30	1	220	1005
		326	766
		427	902
		Average	890
	15	202	----
		351	1121
		457	1024
		Average	1070
	30	196	770
		320	815
		422	938
		Average	840

		$k_f \times 10^{-4}$ cm/sec*		
		Average	Minimum	Maximum
B. Permeability from Pumping Tests on Individual Wells	1-30	1180	905	1445
	6-25	1240	905	1445

* k_f computed from equation 8c with Muskat's correction factor of G = 0.63.

21. Summary. The permeability of the pervious foundation along the central portion of the well system at Trotters was taken to be the average of the permeabilities as computed from the following data:

a. Well data (1943) and piezometer and seepage measurements (1950), $k_f = 1250 \times 10^{-4}$ cm per sec.

b. Drawdown curves from pumping tests on well 15, $k_f = 1070 \times 10^{-4}$ cm per sec.

c. Specific yields obtained from pumping tests on wells 6-25, $k_f = 1240 \times 10^{-4}$ cm per sec.

d. Well flow and piezometer data (1951), $k_f = 1500 \times 10^{-4}$ cm per sec.

The average of the above values indicates that the pervious substratum at Trotters has a permeability of approximately 1250×10^{-4} cm per sec.

Laboratory and Special Pumping Tests

22. In 1953, special pumping tests were performed on several wells in the Mississippi Alluvial Valley about 40 miles south of St. Louis, Mo., to measure the in-situ horizontal permeability of the sand aquifer and of individual sand strata.[54] The wells fully penetrated the alluvial sands. The tests were made for the purpose of accurately determining the permeability of the sand strata for estimating flow from relief wells, depth of screen penetration required to achieve an "effective" penetration of 50% (the penetration assumed in design), and to compare the actual permeability of individual strata with that determined in the laboratory by means of mechanical analyses and/or permeability tests on remolded samples. Similar tests were made on a test well at Old River in Louisiana.

23. The horizontal in-situ permeabilities of the various sand strata encountered at these sites were determined by measuring the flow from each stratum in a fully penetrating well screen by means of a sensitive well flow meter at the boundary of each sand stratum as previously delineated by a boring at each site. The horizontal permeability of each stratum tested was computed from formula 8b for artesian flow into a well (the nature of the formations was such as to cause essentially artesian flow).

24. As a part of these studies, mechanical analyses and laboratory permeability tests were made on samples from different sand strata obtained by means of auger, bailer, and split-spoon samplers, and a 3-in. Shelby tube (undisturbed sampler), and on samples obtained during drilling of the wells. An attempt was also made to correlate the coefficients of permeability as determined in the laboratory and in the field with the effective grain size of the various strata. The tests of the wells also included determination of the specific yield (flow per foot of drawdown in the well) of each well, drawdown in each well

versus time of pumping at a constant rate, drawdown curves to each well at various rates of pumping, flow versus drawdown, and hydraulic head losses through the filter, screen, and in the well. These tests are discussed in the following paragraphs.

Test wells, piezometers, and borings

25. The test wells were 8-in. ID and were of the types and installed by the methods given in table C3.

Table C3

Type of Test Wells and Methods of Installation

Well Number	Type of Well Screen	Filter Thickness in.	Depth of Well ft	Length of Well Screen ft	Sand Aquifer Thickness ft	Sand Aquifer Penetration by Screen %
H-151A	Wood, 3/16-in. slots	6	100	66	95	70
FC-105	Wood, 3/16-in. slots	6	120	86	95	90
OR-1	Metal, No. 12 and No. 8 slots	None	148	99	118	86

Note: Wells H-151A and FC-105 installed by reverse rotary method; well OR-1 with a bailer and casing. Open slot area in wooden screens = 27 sq in. per linear foot of screen.

Fig. C5 shows a section of wooden well screen used in wells H-151A and FC-105. The well flow meter[52] (fig. C5) used for measuring the flow in the well screen consisted essentially of an impeller rotating about a vertical axis, a yoke containing the impeller bearings, an adapter for centering the yoke in the well, a cable for lowering this assembly to any desired depth in the well, and an electrical system for counting impeller revolutions. The meter was calibrated by observing the rate of impeller revolution (determined electrically) at known rates of flow in a vertical 8-in. ID pipe.

26. As holes for the wells were advanced, samples were taken from

the drilling effluent or bailer, depending on the method of installation. With the equipment used, it was not possible to install wells H-151A and FC-105 to the full depth of the sand aquifer because of the presence of large cobbles near the bottom of the alluvial valley. The wells were installed and developed as described in Part VII. The gradation of the filter placed around the screen is shown in fig. C6.

Fig. C5. Well flow meter and 8-in. ID wooden well screen

27. Piezometers to measure the drawdown to the wells were installed in the upper part of the sand aquifer on ranges perpendicular and parallel to the river. (Only data obtained from the latter ranges are included in this appendix.) Some special piezometers were installed at various depths at the outer periphery of the gravel filter around wells H-151A and FC-105 to measure the loss in head through the filter and screen and in the well (see figs. C7 and C8). Piezometers were installed on a line from well OR-1 in both the upper and lower sand strata found at the site of this well.

28. Before installation of any of the wells, an exploratory boring was made at or immediately adjacent to each test well to obtain samples for laboratory tests and to determine the stratification of the sand aquifer. A split-spoon sampler was used to take samples in a mudded hole at wells H-151A and FC-105; logs of these borings are shown in figs. C7 and C8. A 3-in. Shelby tube sampler was used to take undisturbed samples of sand in borings LS-2 and L-8 adjacent to well OR-1 (see fig. C9 for logs of these borings).

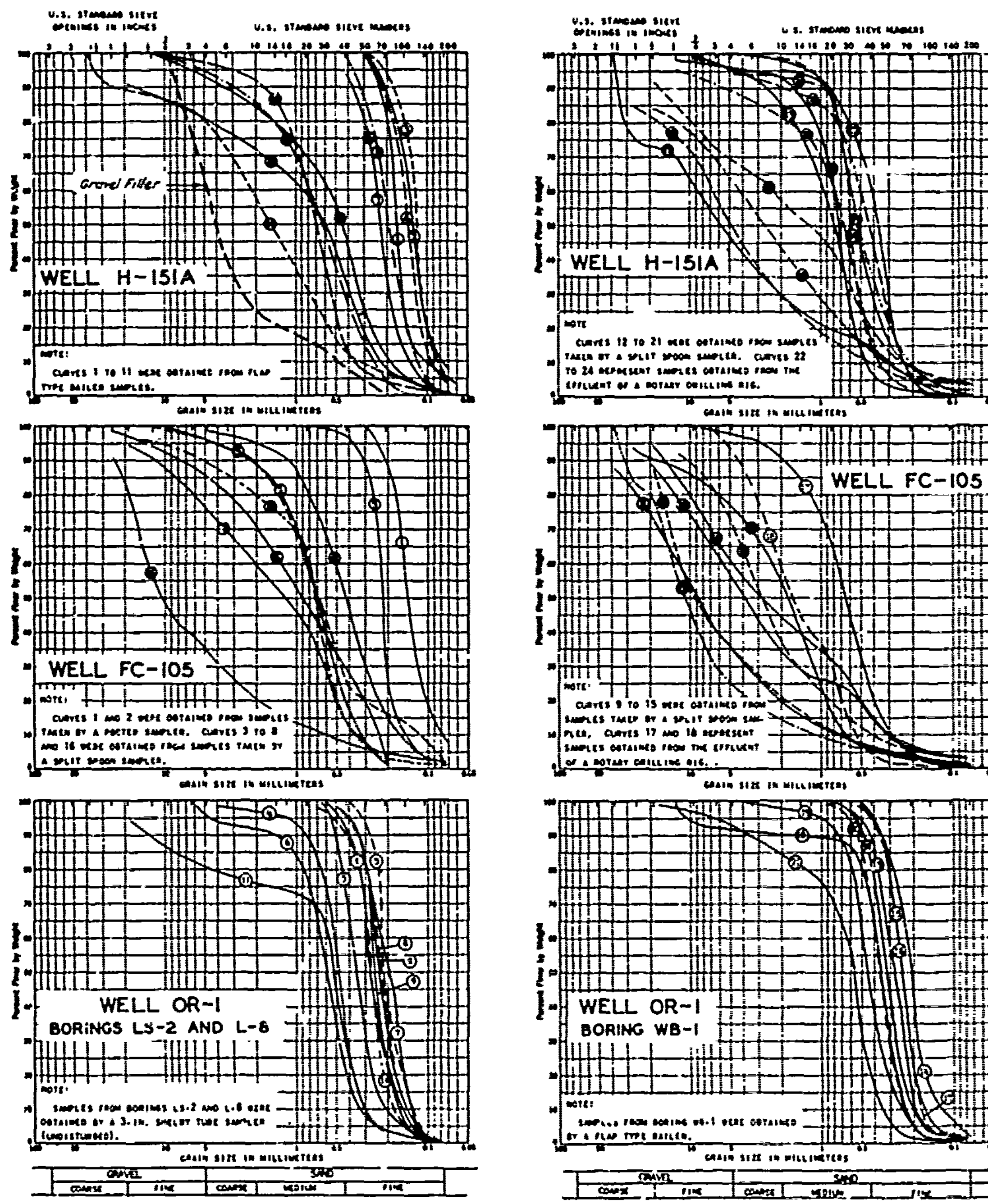


Fig. C6. Gradations of filter gravel and foundation sands

Laboratory tests

29. All samples obtained from the borings and well drilling operations were classified on the basis of the Unified Soil Classification System[49] and are plotted on figs. C7, C8, and C9. Grain-size curves

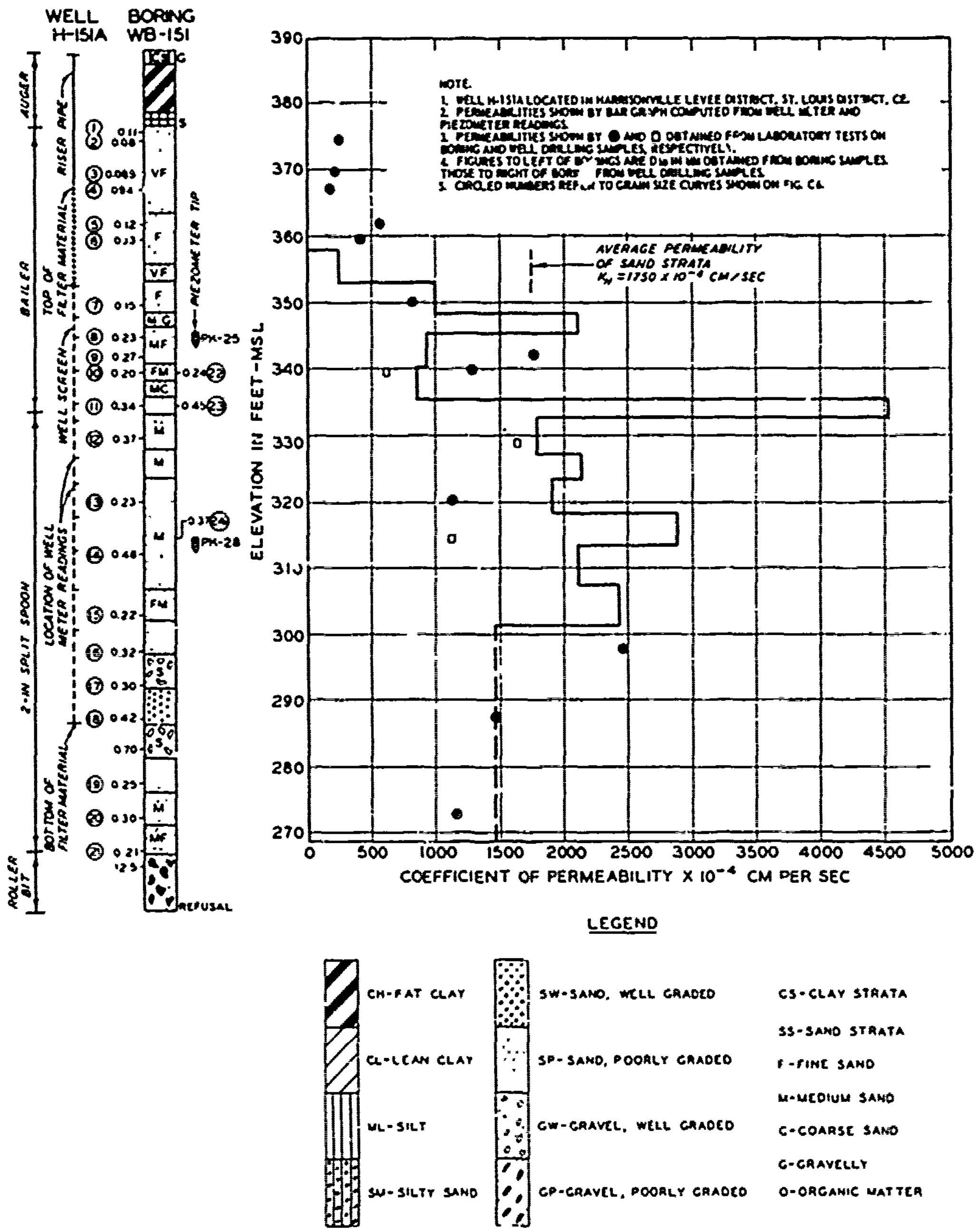


Fig. C7. Coefficient of permeability and effective grain size of individual sand strata, well H-151A

obtained from mechanical analyses of the samples are shown in fig. C6. The reference numbers shown on the grain-size curves correspond to the reference numbers in figs. C7-C9. The effective grain sizes D_{10} of the samples tested are also shown on these figures.

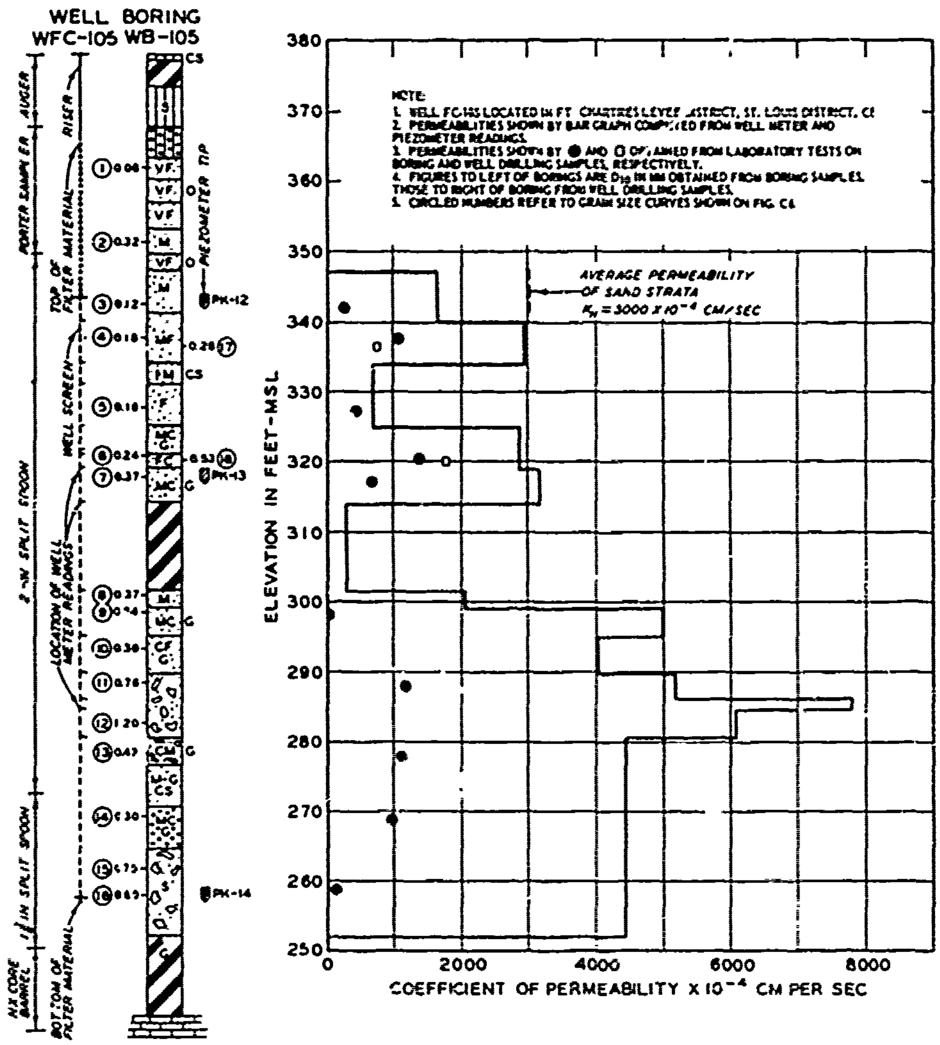


Fig. C8. Coefficient of permeability and effective grain size of individual sand strata, well FC-105

30. The coefficient of permeability was determined in the laboratory on a number of remolded samples obtained by the various sampling methods. The results of these tests are plotted in figs. C7-C9 at the corresponding elevation at which the sample was taken. Samples obtained by means of a split-spoon sampler in a mudded hole were thoroughly washed before testing to remove any traces of drilling mud. Care was taken to insure that no natural "fines" were lost during the washing process.

31. The coefficients of permeability as determined in the

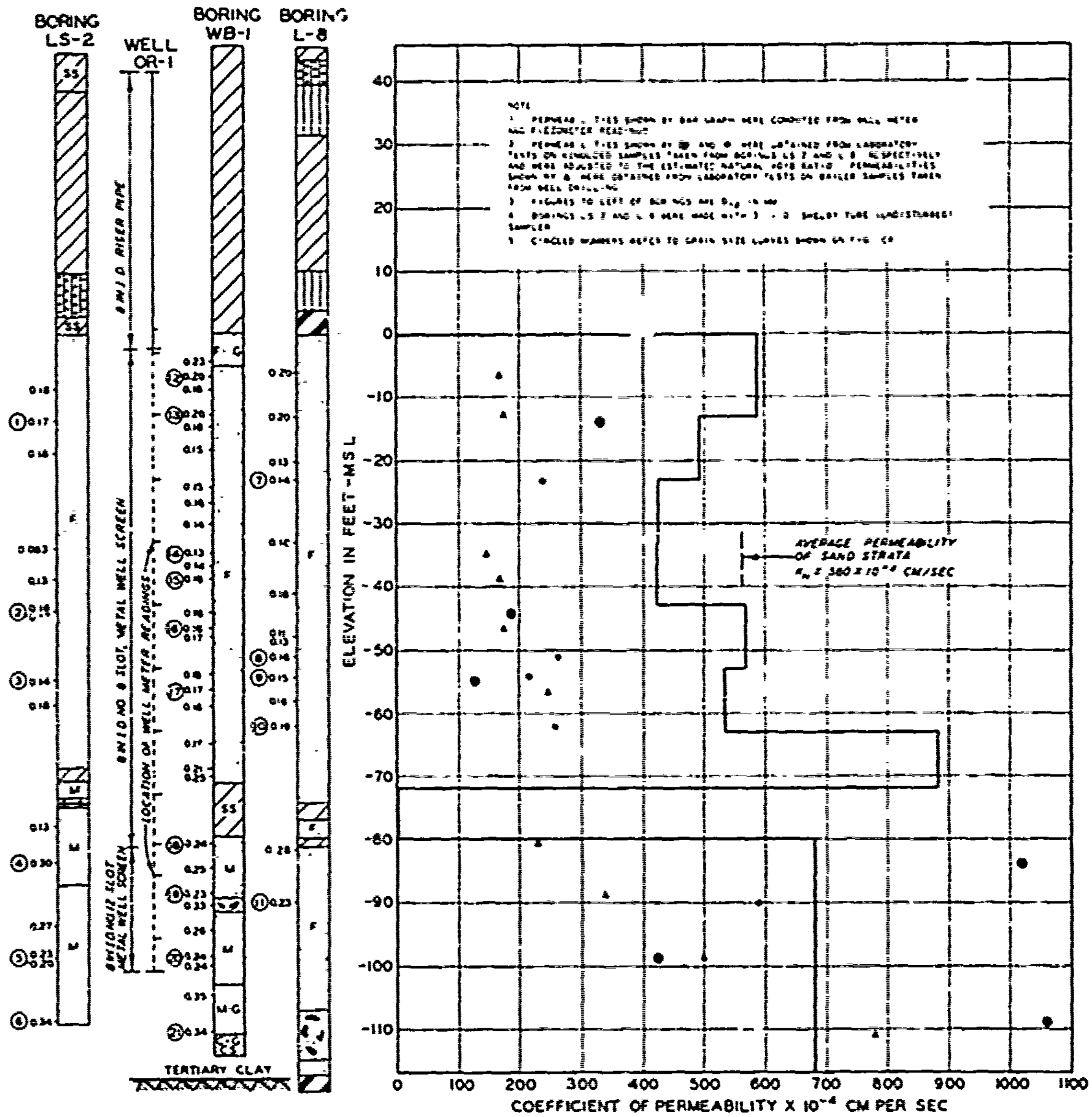


Fig. C9. Coefficient of permeability and effective grain size of individual sand strata, well OR-1

laboratory on samples taken in connection with wells H-151A and FC-105 were adjusted to the estimated natural void ratio for each sample by the formula $k_{R-n} = k_L \times \left(\frac{e_n}{e_L}\right)^2$, and to a temperature of 20 C. The permeability tests on samples obtained from the borings at well OR-1 were run at two or three different void ratios; the permeability at the estimated natural void ratio was interpolated from these data.

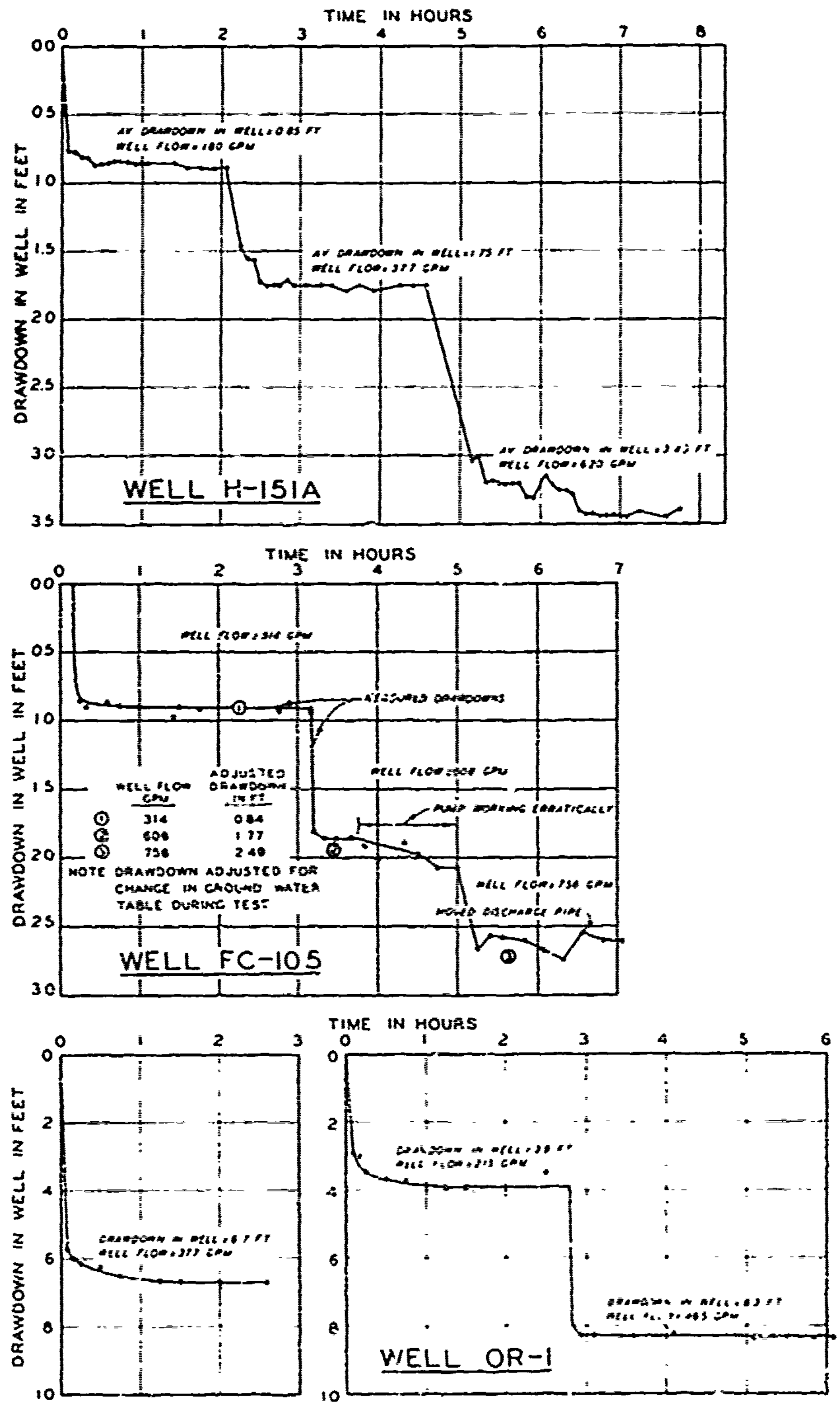


Fig. C10. Drawdown in well vs time for constant pumping rates

The natural void ratio of each sample in situ was estimated from the D_{50} grain size of the sample from the correlation given in fig. 16.

Field pumping test procedures

32. Immediately prior to any of the pumping tests, the ground-water table in the vicinity of each well was determined by reading adjacent piezometers and wells previously installed on a line cut from the test well and perpendicular to the line of seepage flow from the river. The water-surface elevation of the river during the day the test was run was also determined. In general, the river stage was quite stable except during the test on well FC-105 when the river was falling at a rate of one foot a day, resulting in a change in the ground-water table of approximately 0.1 ft per day. The measured drawdowns in the well and piezometers were adjusted to take into account this change in the ground-water table.

33. The pumping tests consisted of pumping each well at three different rates of flow and measuring the drawdown in the well and in adjacent piezometers and wells. An accurate record was kept of the flow and drawdown in the well with time (fig. C10). In pumping the wells an effort was made to set the pump at a constant rate of discharge and then let the drawdown in and adjacent to the well become stabilized. After the drawdown had become stabilized the piezometers were read (fig. C11). The flow in the well screen was also measured at specified intervals or at changes in sand strata by means of the well flow meter. Head losses through the filter and well screen plus those inside the well were measured for wells H-151A and FC-105 by the special piezometers previously mentioned.

Pumping test data

34. The flow to the test wells was essentially artesian because of the existence of upper and lower impervious strata bounding the principal sand aquifer, and the fact that the water in the well was not drawn down below the top of the main sand stratum. As would be expected for artesian flow, drawdown in the test wells stabilized quite rapidly at a constant rate of pumping (see fig. C10); approximately 80% stabilization was achieved in 15-30 min with practically full stabilization in 2 hr.

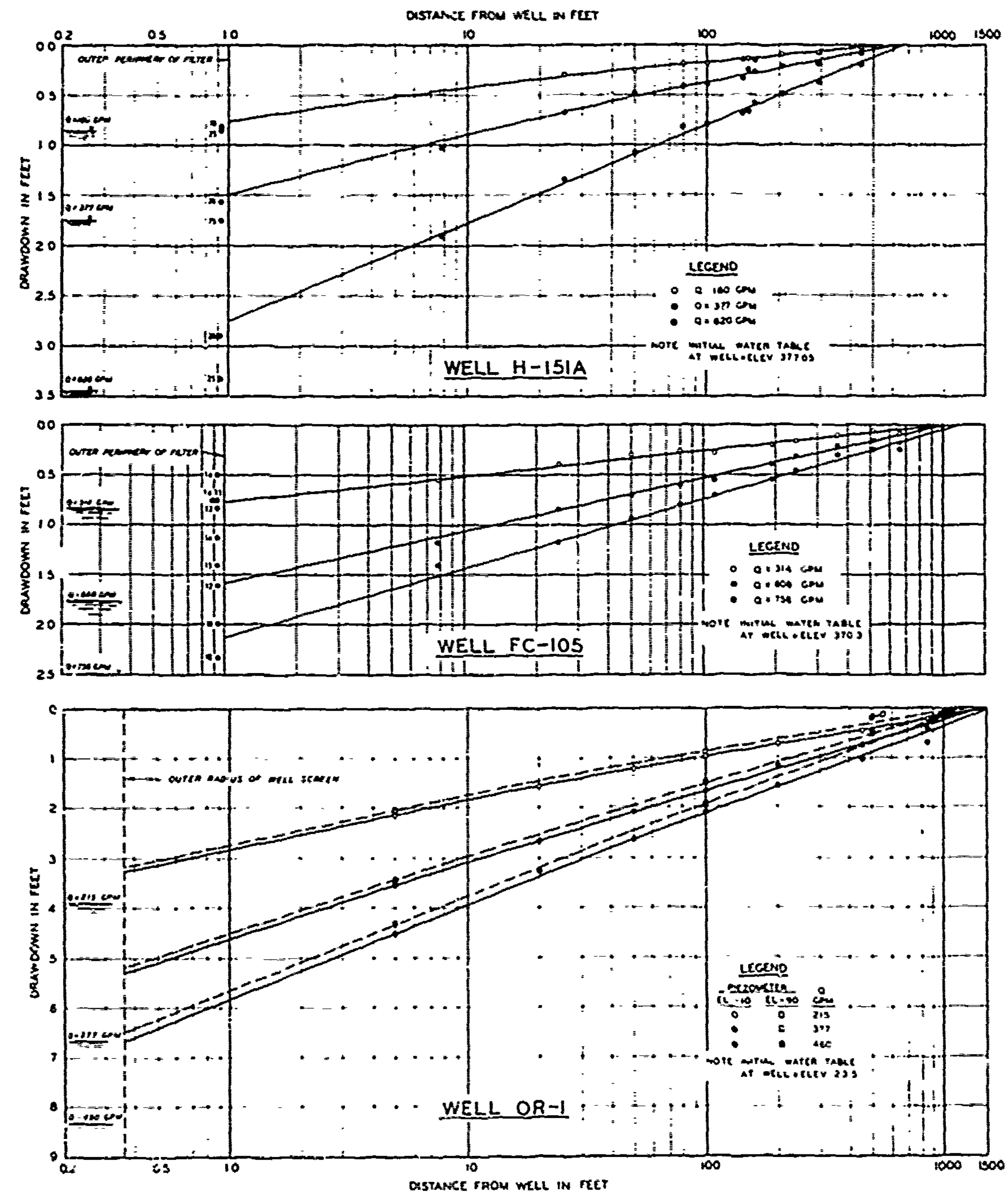


Fig. C11. Drawdown curves

35. Well flow and drawdown within the test wells and in the adjacent piezometers are shown for three constant rates of pumping in fig. C11 together with the elevation of the water table immediately prior to

the start of pumping. As may be noted from this figure, the drawdown curves when plotted on a semilog graph are relatively straight lines. The slight curvature within 20 ft of wells H-151A and OR-1 can probably be attributed to the fact that the screens for these wells did not completely penetrate the pervious aquifer. The intersection of the drawdown curves with the outer periphery of the filter or well screen (for well OR-1) above the elevation of water in the well reflects head losses into and within the well. In this connection it is pointed out that the tips of all of the piezometers except those adjacent to the filter gravel (wells H-151A and FC-105) and at el -90 (well OR-1) were at the upper part of the main sand aquifer. It is of interest to note that the drawdown curves obtained from the deep piezometers at well OR-1 are essentially parallel to but slightly above the curves obtained from readings of the upper piezometers. The vertical difference between the two curves for the same rate of pumping is approximately equal to the head loss inside the well screen between el -10 and -90.

36. The radius of influence was approximately the same for the different drawdowns tested (see fig. C11 and table C4).

Table C4

Radius of Influence of Test Wells

Well Number	Drawdown ft	Distance to Riverbank ft	Radius of Influence Measured Parallel to Riverbank ft	Remarks
H-151A	0.9 to 3.5	1050	550 to 650	---
FC-105	0.9 to 2.6	3000	1000 to 1100	---
OR-1	3.9 to 8.3	900	1400 to 1500	Upper piezometers
			1100 to 1200	Lower piezometers

37. The yields for the test wells are plotted in fig. C12 for various drawdowns. The specific yield or specific capacity of each well is as follows: well H-151A, 221 gpm; well FC-105, 378 gpm; well OR-1, 55 gpm. The drawdown-well flow curves shown for the test wells in

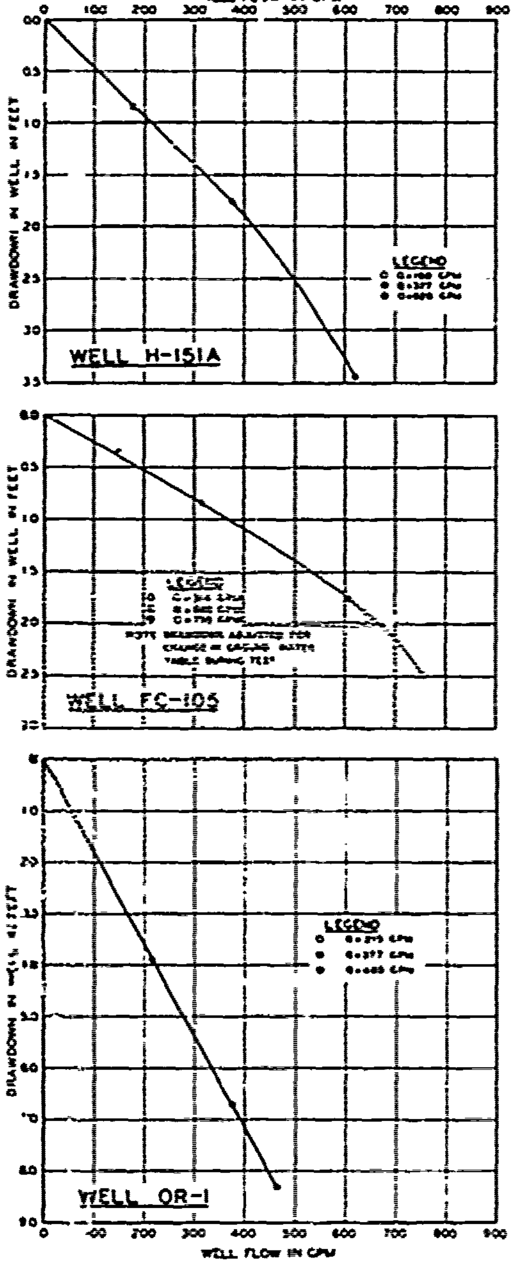


Fig. C12. Drawdown in well vs well flow

fig. C12 are straight lines for well flows up to approximately 400 gpm, thus indicating that the flow to the test wells was essentially artesian. The reduced increase in well flow for corresponding drawdowns for flows in excess of 400 gpm for wells H-151A and FC-105 may be attributed to increased screen entrance losses and hydraulic head losses within the well screen and riser pipe resulting from the high flows.

38. The head loss through the filter and screen as measured by piezometers located at the outer periphery of the filter gravel for wells H-151A and FC-105 is plotted in fig. C13.

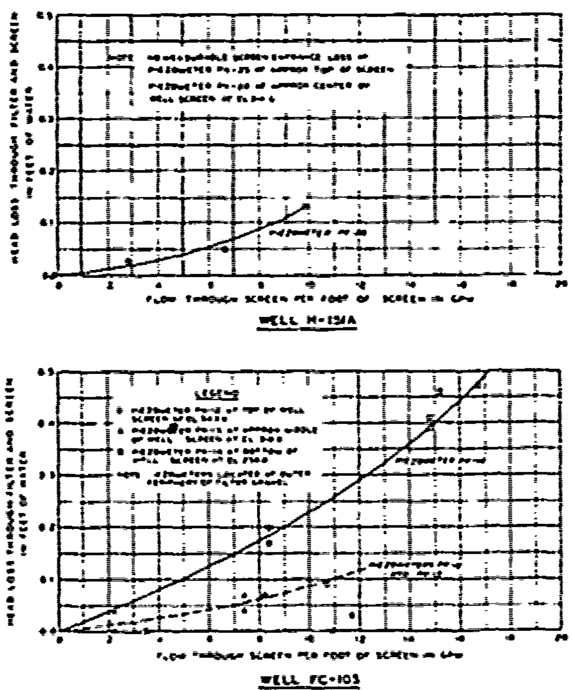


Fig. C13. Head loss through filter and well screen

The flow through the screen at the location of the piezometer tips was obtained from well-flow-meter readings made in the well screen

immediately above and below the piezometer tips. The head loss through the filter and screen as plotted in fig. C13 was taken to be equal to the piezometer readings minus the elevation of the water in the well, minus the computed hydraulic head losses in the well screen and riser pipe from the elevation of the piezometer tip to the bottom of the suction pipe in the well. As may be noted from fig. C13, the head loss through the filter and wooden well screen was quite small, amounting to only about 0.10 to 0.25 ft for a flow through the well screen of 10 gpm per ft of screen. These observed head losses compare quite favorably with head losses measured for a similar well and filter installed in a well testing tank (see fig. C4). The slightly higher entrance loss measured in the laboratory can be attributed to the fact that the laboratory test well had an inside diameter of 6 in. with a total open area of slots of 10.5 sq in. compared to the 27 sq in. per lin ft of screen of wells H-151A and FC-105.

39. The lengths and depths of the screens of the various test wells in relation to the depth and stratification of the sand aquifer in which the wells were installed are shown adjacent to the well log in figs. C7-C9. The locations at which well-flow-meter readings were taken and their relation to the stratification of the sand aquifer are also shown on these figures.

40. The coefficient of permeability of individual sand strata as obtained from the pumping tests and well-meter readings and computed by equation 8b is plotted as a bar graph to the right of the well log in figs. C7-C9. Also plotted on these graphs are the average permeability of the sand aquifer as determined from the pumping tests and the coefficient of permeability as determined from laboratory tests on samples taken from the various sand strata as indicated.

Analysis of pumping test data

41. Too few mechanical analyses and laboratory permeability tests were made on samples of sand from the well drilling effluent from wells H-151A and FC-105 to permit a comparison of effective grain size, grain-size curves, or laboratory permeabilities with samples obtained by split-spoon sampling in adjacent borings. Even if more laboratory tests

had been made, comparisons probably would not have been truly valid because of the high degree of stratification and variation of grain size in the sand strata at the sites for these two wells.

42. At well OR-1 the variation in grain-size characteristics within any specific sand stratum was much less than at the other test wells. In fact, the sands within the same general sand stratum were quite uniform not only with depth but also horizontally between borings which, at this well, were 50 ft apart (see figs. C6 and C9). A comparison between D_{10}'s, laboratory permeabilities, and field permeabilities, based on averages of the data shown in fig. C4 is given in the following table.

Table C5

Comparison of Average Values of D_{10} and k of Samples Obtained with Shelby and Bailer Samplers, Well OR-1

Approximate Stratum Elevation	Boring and Sampler	Grain Size: No. of Samples	Grain Size: D_{10} mm	Coefficient of Permeability x 10^{-4} cm/sec: No. of Samples	k_L Lab	Avg k_H Field
0 to -70	LS-2 (S)*	9	0.15	3	215	---
	L-8 (S)	12	0.15	4	240	560
	WB-1 (B)**	17	0.16	6	180	---
-80 to -103	LS-2 (S)	4	0.25	2	720	---
	L-8 (S)	2	0.25	1	590	680
	WB-1 (B)	3	0.26	3	360	---
-103 to -118	LS-2 (S)	1	0.34	1	1060	---
	L-8 (S)	0	-----	0	-----	680
	WB-1 (B)	2	0.35	1	780	---

* Sample obtained with 3-in. Shelby tube.
** Sample obtained with bailer sampler.

43. As may be seen from table C5 and fig. C9, for the very uniform sand strata at the site of well OR-1 both grain size D_{10} and permeability of sand samples obtained by the two samplers were in agreement. This agreement is attributed to the uniformity of the sand strata, the uniform grading of the sand, and to careful sampling of the material brought out in the bailer.

44. Where the grain size of sands is widely graded, samples obtained with either a piston- or flap-type bailer are not as representative as those obtained with a split-spoon or Shelby tube sampler.

45. Although the permeability measured during the pumping tests varied considerably within a given sand stratum of the same classification, a definite general relationship exists between classification (or D_{10}) and k_H. As would be expected in the alluvial valley of the Mississippi River, permeability generally increased with depth at the sites of the test wells.

46. Although the permeability of the bottom strata in well H-151A (computed from the field pumping tests) was somewhat lower than that of the upper strata, the actual permeability of the bottom strata is probably as high as that of the strata immediately above. The apparent lower permeability may be explained by the fact that the well screen did not completely penetrate the bottom strata for this well and thus flow into this portion of the well screen was less than it would have been if the screen had completely penetrated the bottom strata.

47. The coefficients of permeability determined in the laboratory on remolded samples are plotted at appropriate elevations on the bar graphs of figs. C7-C9. In general, little agreement was found between the laboratory-determined permeabilities of remolded samples and those obtained from the field pumping tests. There is no reason why the permeabilities should agree, particularly where the aquifer is stratified and lenses of coarse sand and fine gravel exist. Generally, for any given stratum, the field permeabilities exceeded the normally determined laboratory permeability by two to four times.

48. Plots of effective grain size versus coefficients of permeability as determined in the laboratory on remolded samples and from pumping tests on approximately 100% penetrating well screens are shown in fig. C14. (The D_{10} used in the plot for the pumping test is the average D_{10} for the specific stratum tested.) A great deal of scatter of the points is apparent for both sets of curves. No attempt was made to relate the permeability as measured in the laboratory or field to the uniformity or shape of the grain-size curve. Some of the widely

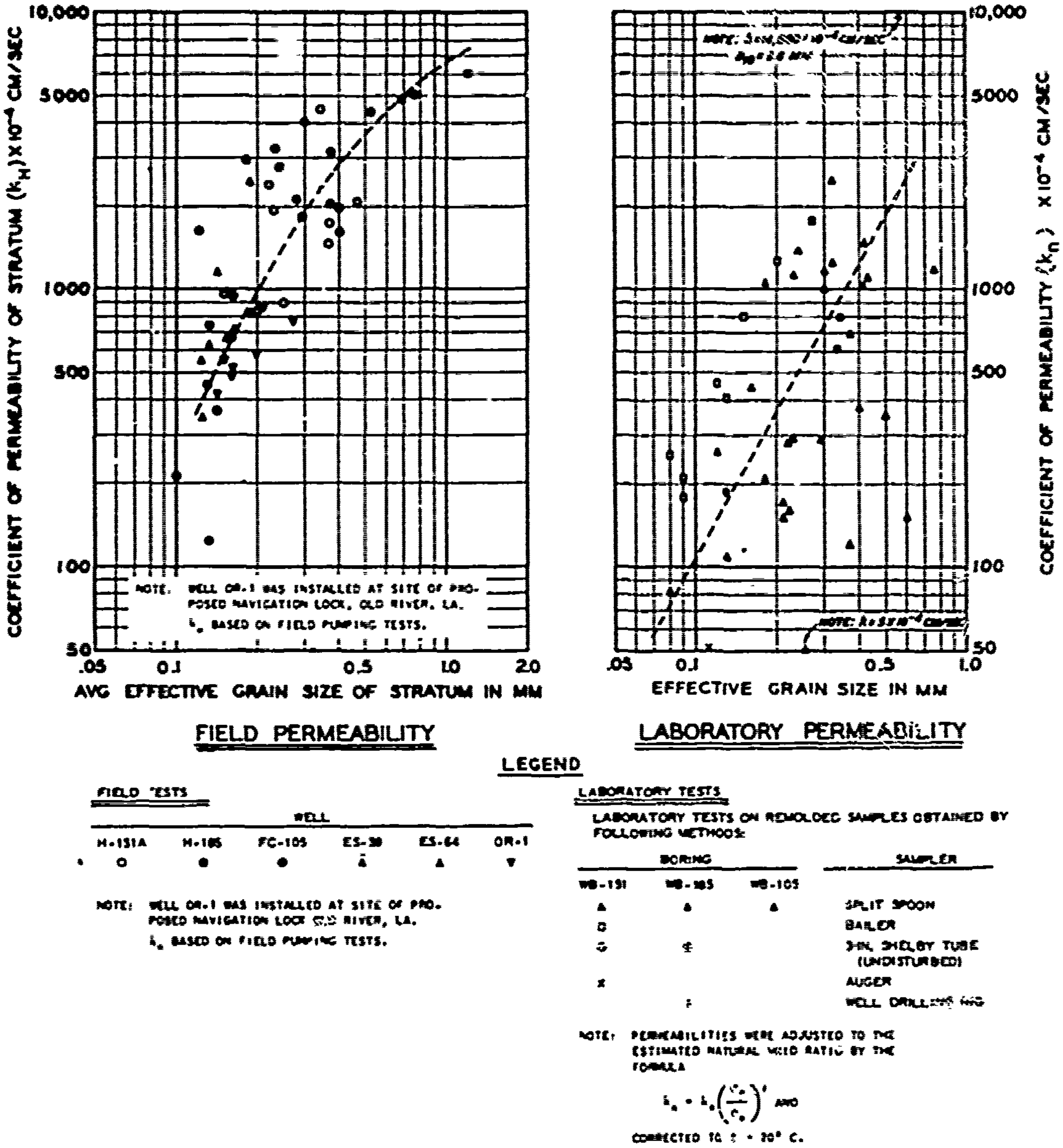


Fig. C14. Summary of coefficient of perme. bility vs effective grain size

graded grain-size curves plotted in fig. C6 really represent a mixture of alternating strata of fine or medium sand and coarse sand or fine gravel, and such deposits in their natural state may have horizontal coefficients of permeability that are quite high.

49. The graphs in fig. C14 are not considered particularly accurate but the graph showing the relation between D_{10} and field

permeability k_H may be of use, in the absence of pumping test data, to engineers concerned with seepage, water supply, or irrigation problems in the Mississippi River Valley.

Summary

50. Laboratory tests on remolded sand samples, regardless of how the sample was obtained, do not give reliable coefficients of permeabilities for estimating the seepage or water-carrying capacity of sand strata in the Mississippi Valley.

51. As far as grain-size characteristics go, reasonably reliable samples can be obtained from uniform sand deposits with either a Shelby tube, split-spoon sampler, or bailer, provided the sampling is properly done. However, some sand grains smaller than a No. 100 sieve may be lost in bailer sampling. For stratified deposits or where the sands are more widely graded, a Shelby tube or split-spoon sampler should be used.

52. For determining the permeability of individual strata of a sand aquifer the most accurate procedure is considered to consist of a pumping test of a well with the screen penetrating all of the sand strata, and measurement of the flow within the well screen with a well flow meter at strata changes, as described in this appendix. The changes in strata may be determined by a nearby boring in advance of the test, or logged as the hole for the well is advanced.

53. The permeabilities of sand aquifers in the Mississippi River Valley may be reasonably estimated from the relation between D_{10} and k_H given in fig. C14.

APPENDIX D: FIELD STUDY OF CONTROL OF UNDERSEEPAGE BY RELIEF WELLS

1. A part of the general study of methods for controlling underseepage along levees in the Lower Mississippi River Valley included the installation and testing of a full-scale relief well system at Trotters 54 (see plates 84 and 87). A full report on the design, installation, and testing of this system has previously been published.[51] However, in view of the importance of this particular study to the subject of underseepage and its control, it is summarized in this appendix.

2. The primary purposes of installing the new relief well system at Trotters were to make a full-scale field test of the efficacy of a relief well system and to obtain more knowledge concerning the action of relief well systems in general. Other purposes of the study were:

- a. To determine the distribution and amount of hydrostatic pressure in the pervious substratum with and without relief wells in operation, including the head between the wells and landward of the well system.
- b. To estimate the "effective" source of underseepage and the amount of underseepage with and without relief wells in operation.
- c. To determine the effect of different well spacings on the reduction of substratum pressure and well flow.

3. Basis for selection of the site at Trotters 54 for an experimental relief well installation, geology and foundation conditions, history of underseepage, and the source of seepage at the site are discussed in Part IV.

Observed Hydrostatic Pressure

4. Piezometer data obtained during the 1950 high water, before wells were installed at the site, are shown on plate 97. During this high water the river stage reached an elevation of about 13.8 ft above the average ground surface landward of the levee and 15.7 ft above the water elevation in the drainage ditch paralleling the toe of the seepage berm. The maximum hydrostatic pressure observed at the toe of the seepage berm

was 3.0 ft above the average ground surface or 4.9 ft above the tailwater in the drainage ditch, or 35% H and 50% H, respectively.

5. At the 1950 crest, excess pressures above the ground surface were observed as far as 3500 ft landward of the toe of the seepage berm. Data obtained from a line of piezometers perpendicular to the levee indicated that most of the seepage passing beneath the levee was rising to the surface in a strip about 300 ft wide along the toe of the seepage berm.

Design of Well System and Appurtenances

Well system

6. Criteria of design. The experimental well system at Trotters was designed for two different cases. Case A was based on the assumption that all flow from the well system either would be pumped over the levee or that natural flow conditions would be such that the water in the collector ditch would not rise above el 178 and that the artesian head landward of the levee would not rise above the ground surface (el 180). Thus, little natural seepage would develop landward of the wells and it could be assumed that the landward top stratum would be analogous to an impervious top stratum case. For this case the maximum allowable head between wells P would be 8.4% H.

7. Case B was based on the assumption that the flow from the wells would not be pumped and that the tailwater over the wells would be at el 180 and that the net head between wells would not be more than 2 ft above the natural ground surface, or 7.2% H. In this case the top stratum landward of the wells was assumed to be only relatively impervious

$$\left[k_b = 2 \times 10^{-4} \text{ cm/sec}\right] .$$

8. Most formulas and model test data available for the design of relief well systems are based on an infinite line of wells. In the design of the Trotters relief well system it was decided to make the system only long enough (1450 ft) to provide a central section of about 1000 ft which

would be affected relatively little by end effects. The well system as finally designed consisted of 30 wells spaced on 50-ft centers.

9. Bases of design. The following assumptions and values were used in designing the Trotters relief well system:

a. Effective well radius $r_w = 10$ in. $= 0.833$ ft.

b. Infinite line source of seepage, parallel to an infinite line of relief wells, assumed in computing well flow and head between wells. End effects were considered in computing total flow from system.

c. Distance from source of seepage to line of wells $s = 900$ ft.

d. Thickness of top stratum $z_b = 10$ ft.

e. Thickness of pervious stratum $d = 90$ ft.

f. Permeability of pervious stratum $k_f = 1250 \times 10^{-4}$ cm/sec.

g. Penetration of well screen based on 50% penetration of principal pervious aquifer.

10. The formulas and model data used in computing the performance of the Trotters well system for case A were Jervis' design curves[23] and WES model A-a-1. Computation of the performance of the system for case B was based on WES model A-a-2. These design curves and model data are presented in Part VI of the main report and/or in Appendix A. All model data were adjusted to the foundation conditions existing at Trotters and the wells used at Trotters.

11. Design computations. All values given in the following paragraphs pertaining to computation of well flow, landward pressure, and seepage are based on an infinite line of wells unless otherwise stated. The performance of the Trotters well system was computed for well spacings of 50 and 100 ft and a screen penetration of 50% as described below.

12. Case A. For $W = 50\%$ and $a = 50$ ft, Q_w was computed from Jervis' curves to be 9.0 gpm per ft H and 9.0 gpm from WES model A-a-1 adjusted to the Trotters site. P was computed from Jervis' curves to be 4.1% H and 6.2% H from WES model A-a-1.

13. Case B. Well flow, natural seepage (no wells), natural seepage (with wells), and head between wells all as computed from WES model A-a-2 are as follows:

$Q_w = 7.6$ gpm per ft H . $P = 6.2\%$ H

Natural seepage (no wells) $\frac{(Q_s)}{H}$ = 6.6 gpm per 50 ft of levee

Natural seepage (with wells) $\frac{(Q_{s-w})}{H}$ = 1.0 gpm per 50 ft of levee

Total flow (well plus seepage), Q_{s+w} = 8.6 gpm per ft H per 50 ft of levee

Increase in total flow due to wells = 29% .

14. The design curves for head between wells and well flow are shown for cases A and B on fig. D1. The curves for P/H do not include any screen or hydraulic head losses in the well. The performance of the wells near the ends of the system was expected to be influenced by end effects but the computed Q_w and P/H were considered applicable to the central portion of the system.

15. Screen entrance and hydraulic head losses in the well were

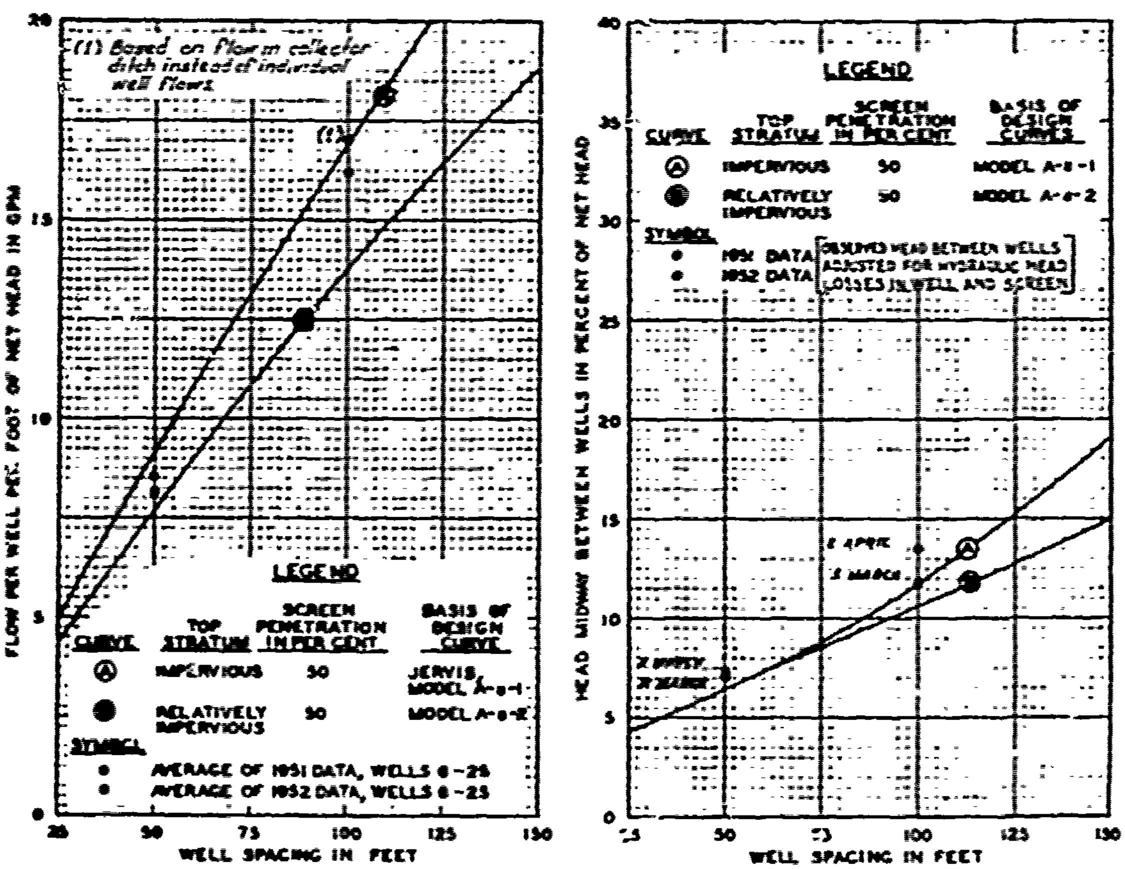


Fig. D1. Well flow and head between wells

computed from fig. D2 and added to the computed head between the wells as follows:

	Case A	Case B
Head on well system for project flood	29.7 ft	27.7 ft
Well flow, Q_w	300 gpm	240 gpm
Computed P	2.1 ft	2.0 ft
Entrance and well loss, h_w	0.9 ft	0.5 ft
Total head between wells (project flood)	3.0 ft	2.5 ft
Head between wells	7 to 10% H	7 to 9% H

16. Well screen, gravel filter, and riser pipe. The well screens consist of 6-in.-ID redwood pipe slotted with 3/16- to 1/4- by 2-in. slots. The slots have an open area of 18 sq in. The screen portion of each well is 40 ft long. It is surrounded with a 6-in. layer of filter gravel (see fig. D3) extending from 1 ft below the plug in the bottom of the screen to 1 ft above the top of the screen. The design of this filter was primarily based on the filter criterion

$$\frac{D_{15}\text{ filter}}{D_{85}\text{ sand}} < 5.0 .$$

The filter has an estimated permeability of about 5000×10^{-4} cm/sec. The minimum D_{85} of the filter is 1.2 times the slot width of the well screen. Details of the well are shown in fig. D4.

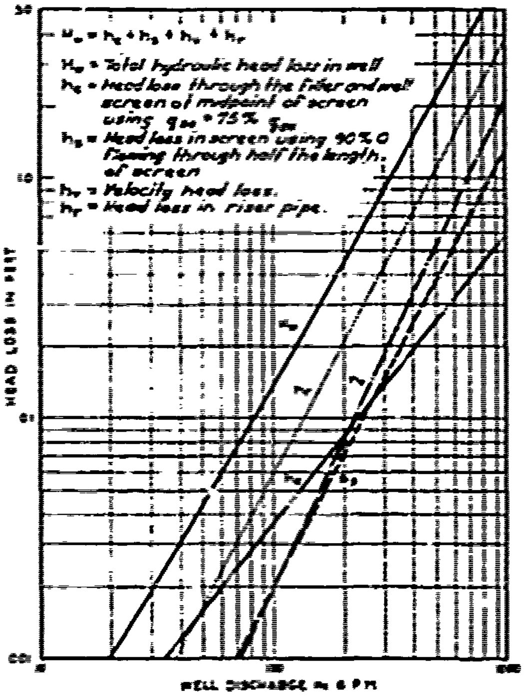
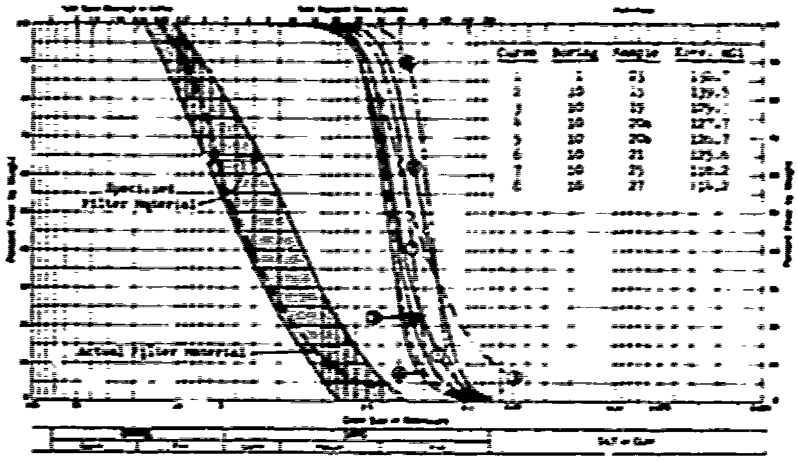
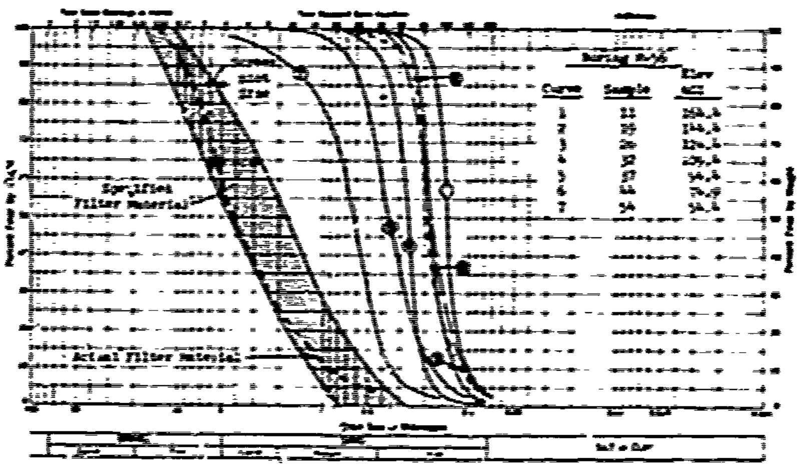


Fig. D2. Computed head losses in wells for natural flow condition

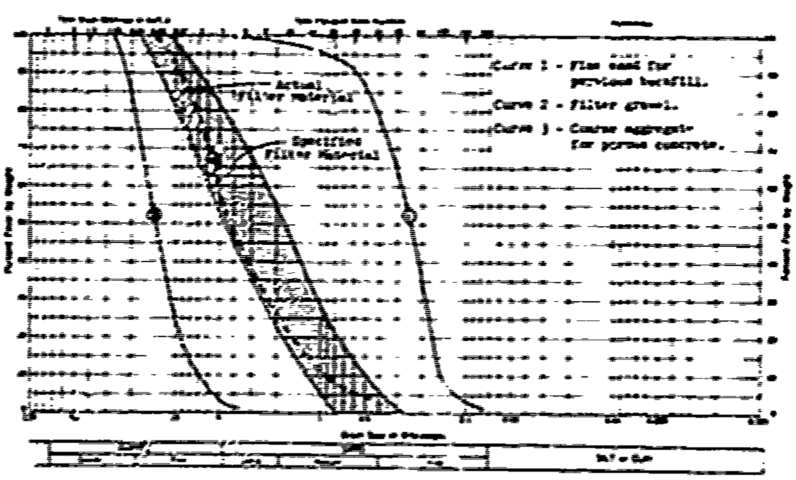
17. The tops of the riser pipes are capped with brass flap valves which may be used to close the wells or throttle the flow if desired. The wells are kept closed when there is no head against the levee.



BORINGS 1 AND 10



BORING M-58



PERVIOUS BACKFILL, FILTER GRAVEL, POROUS CONCRETE AGGREGATE

Fig. D3. Foundation sands and filter gravel

Appurtenances

18. Collector ditch. The collector ditch for the wells is about 3.5 ft deep and has a bottom width of 3 ft. It is paved with porous concrete underlain with a 2-in. blanket of well-graded coarse sand (see fig. D4). The paving has a thickness of 6 in. in the bottom of the ditch and 4 in. on the side slopes. The ditch has a capacity of about 35 cfs.

19. Control culvert. A control culvert through the sublevee at the downstream end of the well system (plate 83) consists of a 30-in. corrugated pipe and a 101 Calco gate. The culvert will pass the total estimated maximum flow from the well system, plus the runoff from the levee into the sublevee basin, without overflowing the sublevee. The control culvert will pass the maximum well system flow for case B, Q_w = 17 cfs, with a head of 0.5 ft or water elevation at the middle of the system of 180.7.

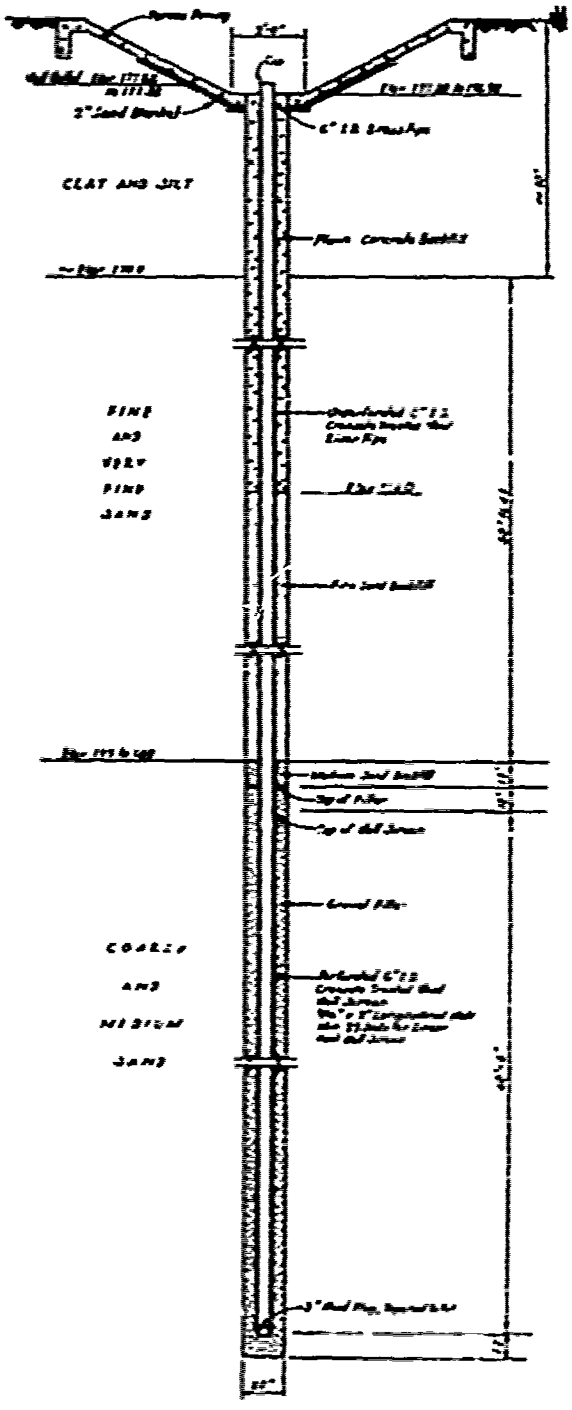


Fig. D4. Relief well and collector ditch

Installation of Wells and Pumping Tests

20. The holes for the wells at Trotters were made by the reverse-rotary method as described in Part VII.

21. After development of the well by surging, a pumping test was run to determine the flow for various drawdowns in the well. The well flows for these drawdowns usually ranged from 200 to 500 gpm. No sand was observed in the effluent from any of the wells during the pumping tests.

Analysis of Well Flow and Piezometer Data

22. Operation of the relief well system was observed during high-water periods in 1951 and 1952. The maximum head on the well system was about 7.5 ft in 1951 and 10.2 ft in 1952. During both high waters the well system was operated on both 50- and 100-ft centers; also, during each high-water period the wells were completely closed and substratum pressures and seepage flows were measured. Piezometer readings and well flows were observed at frequent intervals for different well operating conditions.

23. The general operating procedure was to allow the system to operate with the wells on 50-ft centers up to about crest stage, at which time every other well was closed so as to create a system with wells on 100-ft centers; after obtaining well flow measurements and piezometer readings for both of these conditions, the well system was closed and piezometer readings and seepage measurements made. After obtaining these measurements, all wells were then reopened.

Well flow

24. The individual well flows were measured by a special well flow meter which could be lowered into the wells. The flow in the collector ditch was measured at the downstream end by cross sectioning the stream and measuring the velocity of flow by means of a midget Gurley flow meter.

25. The average of individual well flows per foot of net head for all data obtained during rising river stages during the 1951 and 1952 high-water periods is shown by fig. D5. (Well flows for a = 100 ft during 1952 high water are not shown because well meter was apparently not functioning properly.) The average flows per foot of net head on the

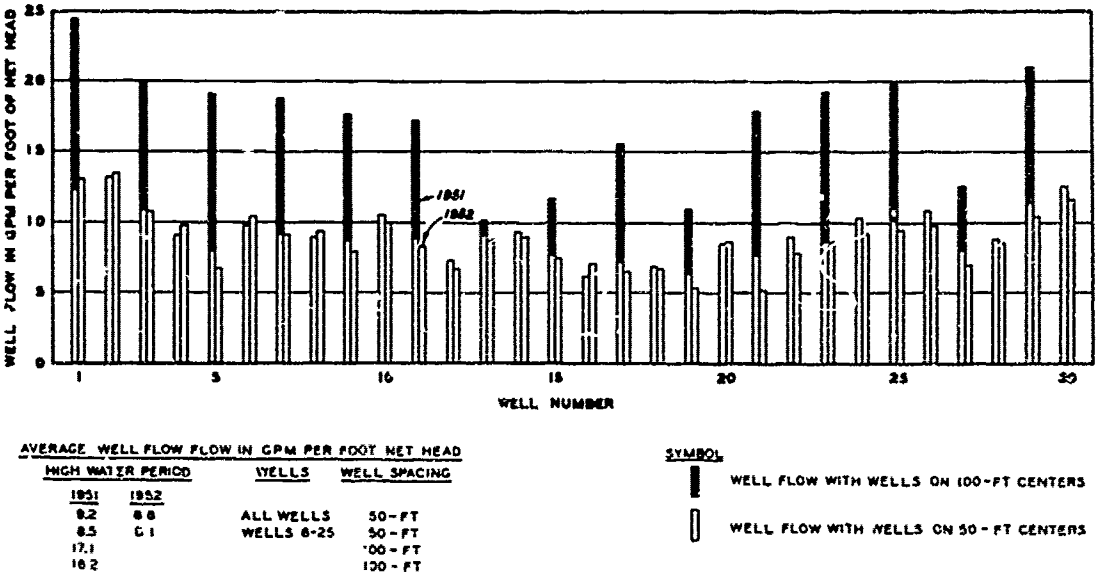


Fig. D5. Individual well flow per foot of net head (1951 and 1952)

system for wells 6 through 25 are also tabulated in fig. D5 for well spacings of both 50 and 100 ft. Well flows in the center of the system for a 50-ft well spacing averaged about 8.5 gpm in 1951 and about 8.1 gpm per well in 1952 and compare fairly well with the predicted flow of 9.1 gpm. The average individual well flow per foot of net head with the wells on 100-ft centers was 16.2 gpm in 1951 and 11.3 gpm in 1952 compared to the predicted flow of 16.8 gpm. It is pointed out that on 2 April 1952, with the wells on 100-ft spacing, the collector ditch flow exceeded the sum of the well flows by 50%. This discrepancy is attributed to improper functioning of the well flow meter for this test. If the flow measurements in the collector ditch were correct, then the average flow from the wells on 100-ft centers would be about 16.9 gpm per foot of net head on the system compared to the predicted flow of 16.8 gpm. The bar diagrams on fig. D5 show that in general the wells at the end of the system discharged more than the wells in the central portion of the system. This trend is attributed to increased flow caused by end effects.

26. The relationships between well flow and net head on the well system are shown in fig. D6 for the 1951 and 1952 high-water periods. It can be seen that the 1951 and 1952 well flows are similar up to a net head

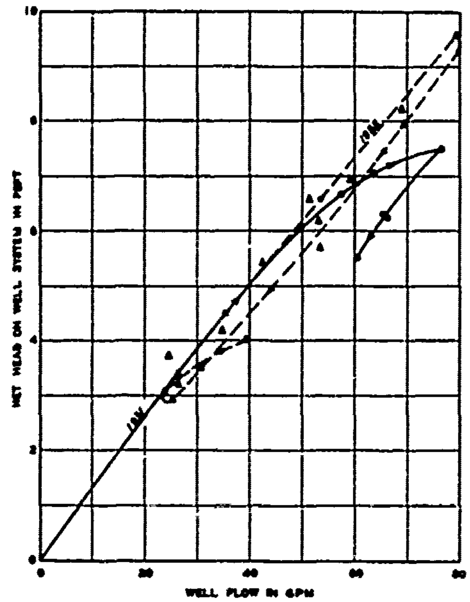


Fig. D6. Average individual well flow, a = 50 ft (1951 and 1952)

of about 5 ft, and that the well flow varies linearly with net head up to about 5 ft in the well system. For net heads greater than 5 ft, the 1951 well flows increased at a rate greater than the rate of net head increase; and after the crest of the flood had occurred, greater well flows were obtained at a given head than the flows occurring under the same head before the crest had occurred. The 1952 data indicate a nearly linear relationship between flow and net head for the heads experienced during rising river stages. As in 1951, after the crest of the flood had occurred, greater flows resulted under the same head. Inasmuch as the flow from the relief well system should vary linearly with the net head on the system, provided the distance to the effective source of seepage remains constant, the occurrence of a greater flow under a given net head after the crest of the flood has passed is attributed to a decrease in the distance to the effective source of seepage.

27. Flow from the well system was carried by the outfall ditch, located at the end of the system and landward from the levee, without overflowing at any point during both the 1951 and 1952 high-water periods.

Substratum pressure

28. Piezometer gradients along line M for the 1952 high-water period with and without the relief well system in operation are shown in fig. D7. The piezometric data plotted on this figure show that very little substratum pressure developed along or landward of the line of wells when all wells were open. Only in a few isolated areas landward of the well systems was there any excess head above the ground surface. A prime reason for there being no head above the ground surface was that the wells discharged at an elevation below the natural ground surface, and no ponding occurred in the outfall ditch which conducts the well flow away from

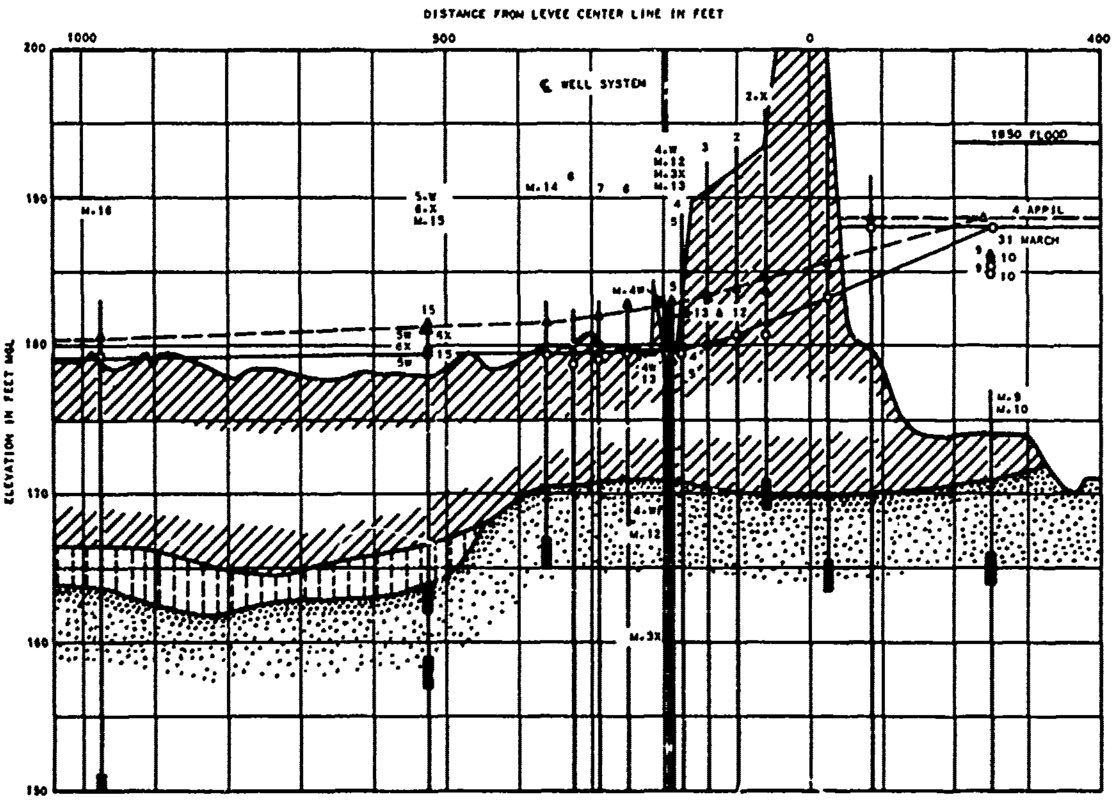


Fig. D7. Piezometer gradients, line M, 1952

the levee. When the wells were closed, the substratum pressures rose rapidly, and the net hydrostatic pressure in the substratum was between 40 and 50% H.

29. River stages, the hydrostatic pressure along the line of wells when closed, the average ground surface and the piezometric surface between wells for spacings of 50 and 100 ft, and the elevation of the water surface in the collector ditch all are shown for certain selected days during the crest of the 1952 high water in fig. D8. The head midway between wells along the system as shown in fig. D8 was plotted in fig. D9 for well spacings of 50 and 100 ft. The observed data shown in this figure were corrected for well losses.

Seepage control

30. The efficacy of the relief well system in controlling seepage was demonstrated by the absence of sand boils and/or excessive seepage landward of the well installation during the 1951 and 1952 high-water

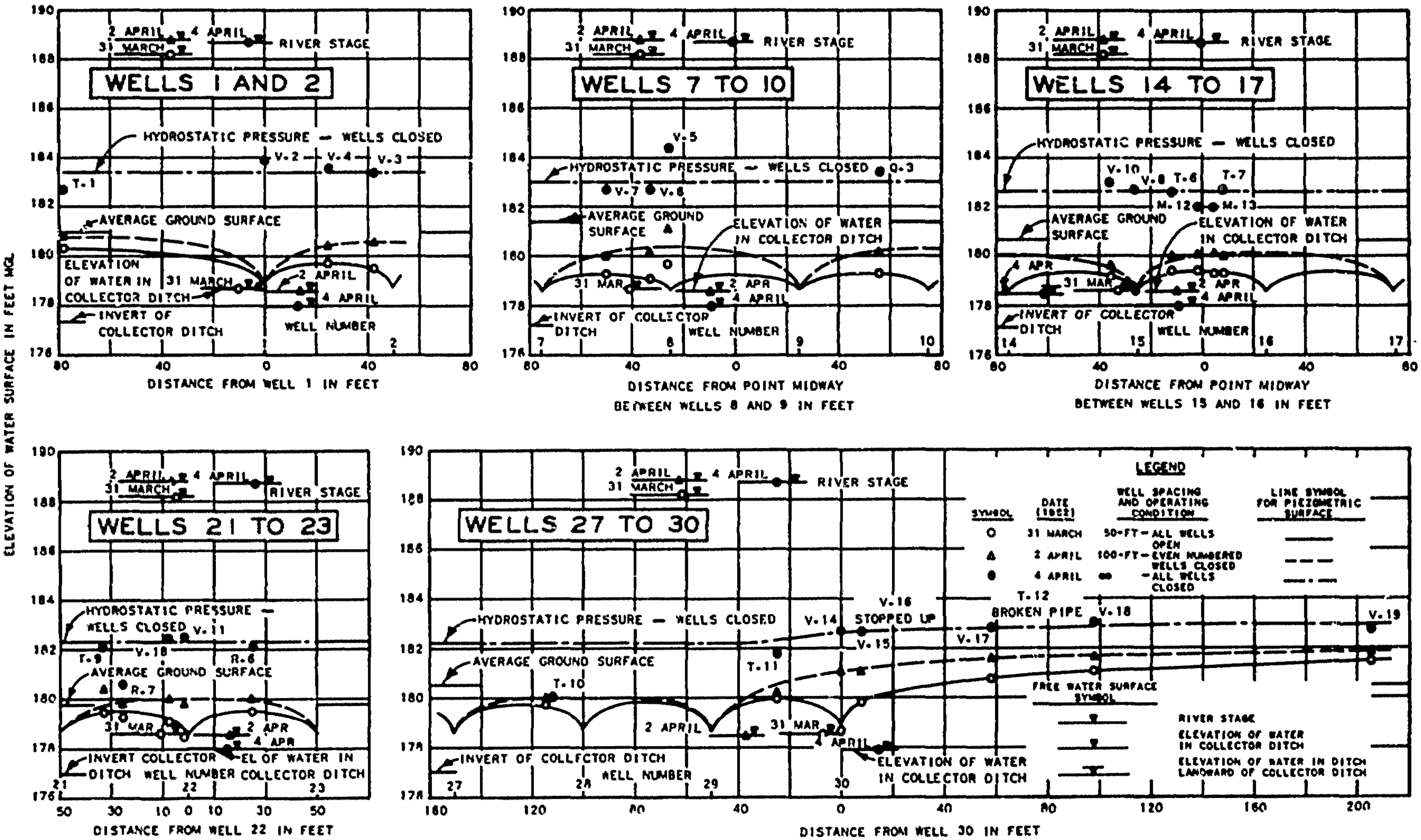


Fig. D8. Hydrostatic head between wells (1952)

periods. Again it should be noted that the efficiency of the system is increased greatly by virtue of the landside drainage conditions and because the wells discharge at an elevation somewhat lower than the natural ground surface. A comparison of seepage conditions both upstream and downstream from the well system further indicates the ability of the system to control seepage. The following summary of seepage conditions was abstracted from field notes taken during operation of the well system in the two high-water periods.

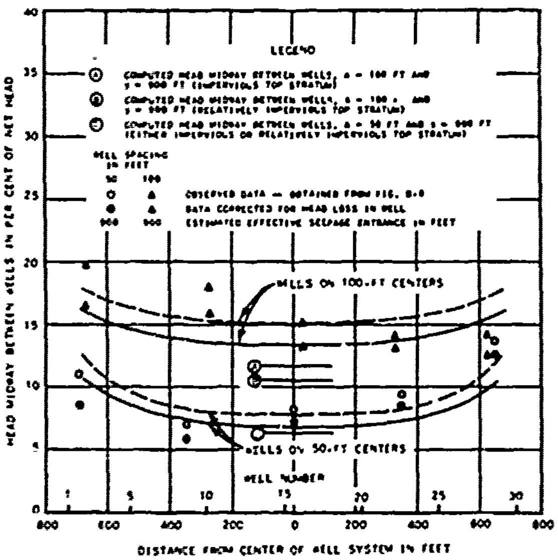


Fig. D9. Head midway between wells along well system

31. While the well system was in operation with wells on either 50- or 100-ft centers no sand boils were observed in the drainage ditch landward of the levee toe adjacent to the well system; however, inspection of this ditch upstream and downstream from the well system disclosed the presence of several pin boils in the bottom of the ditch for distances of about 1000 ft upstream and downstream from the ends of the well installation. No seep water was observed landward of the well system when it was in operation but seep water was observed standing in a field 600 to 900 ft landward of the levee toe at a point about 500 ft downstream from the relief well system. These conditions were observed for river stages from about 5 to 11 ft above the average ground surface.

32. After the river crested at a stage of approximately 10.5 ft above the elevation of the water surface in the landside ditch in 1952, all relief wells were closed. Numerous sand boils became active in the outfall ditch for a distance of 1100 ft landward of the installation and in the drainage ditch paralleling and landward of the levee. Seepage into the ditch was 18.2 gpm per 100 ft of levee, or about 11.5 times that which had occurred immediately before when the wells were operating on 100-ft

centers. Upon reopening all the relief wells (with H = about 9.3 ft) sand boils previously noted in the outfall and landside drainage ditches became inactive and no seepage was noted along the sublevee or flowing into the drainage ditch landside of the levee.

33. It is apparent from these seepage observations that the well system is effective in controlling sand boils and seepage landward of the well system for the stages experienced. The piezometer readings together with the observations made during the 1952 high water indicate that the well system not only reduced the substratum pressures but also intercepted a large portion of the natural seepage which otherwise would have risen to the surface landward of the levee.

34. Some seepage and minor sand boils must be expected for the project flood; but if the system is maintained in good operating condition, underseepage during the project flood should create no problem along the reach of wells.